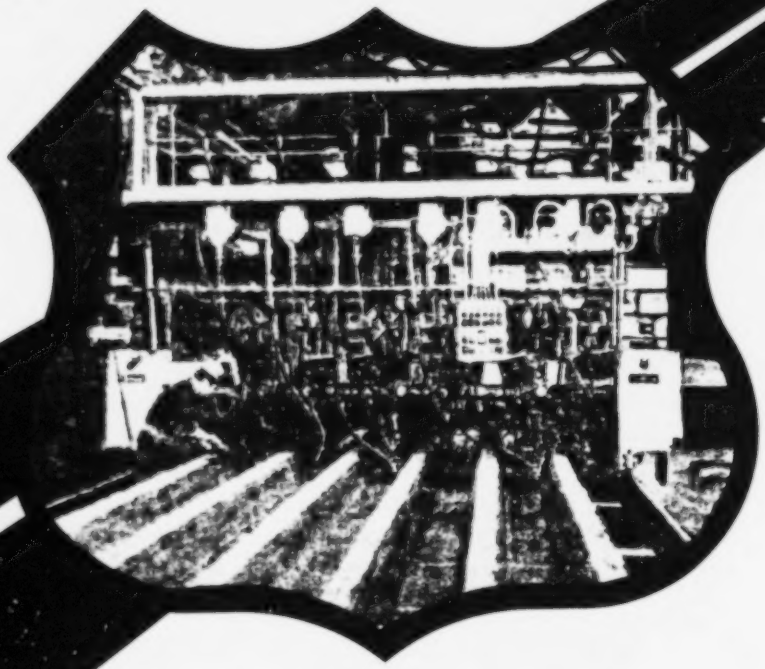
JUL 8 1982

TD 2.30: 81/125

Report No. FHWA/RD-81/125

ASSESSMENT OF CURRENT FABRICATION TECHNOLOGY FOR WELDED BRIDGES

March 1982
Interim Report



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Prepared for

U.S. Department of Transportation
Federal Highway Administration
Offices of Research & Development
Structures and Applied Mechanics Division
Washington, D.C. 20590

Document is available to the U.S. public through the National Technical Information Service Springfield, Virginia 22161

FOREWORD

This report presents a general overview of welding procedures and practices followed in bridge fabrication. Research in improved fabrication and inspection of welded connections for bridges is part of the Federally Coordinated Program, Project 5L, "Safe Life Design for Bridges," Task 1, Fracture and Quality Control for Improved Structural Performance.

It is anticipated that the information contained in this report will aid the bridge engineering community in determining the most economic and reliable welding processes for steel bridge fabrication.

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Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

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Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/RD-81/125		

4. Title and Subtitle	5. Report Date
Assessment of Current Fabrication Technology for Welded Bridges	March 1982
	6. Performing Organization Code: A1-29
7. Author(s): R. N. Johnson and others	8. Performing Organization Report No.: 001
9. Performing Organization Name and Address: GARD, Inc. A Subsidiary of GATX Corp. 7449 N. Natchez Ave. Niles, Illinois 60648	10. Work Unit No. (TRAIS): 35L1-032
	11. Contract or Grant No.: DTFH61-80-C-00083
12. Sponsoring Agency Name and Address: Offices of Research and Development Federal Highway Administration U. S. Department of Transportation Washington, D. C. 20590	13. Type of Report and Period Covered: Interim Report Tasks A and B August-December, 1980
	14. Sponsoring Agency Code

15. Supplementary Notes

FHWA Contract Manager: Charles H. McGogney (HRS-11)

16. Abstract

This report is a general survey which evaluates all the pertinent work that has been accomplished in recent years in the area of bridge welding. It includes discussions involving base material selection and weldability studies, specific welding techniques employed, governing codes and specifications, metallurgical aspects including microstructure and mechanical properties, inspection techniques - nondestructive evaluation, weld flaw studies, and examples of successful bridge welding in other countries. This information will aid in determining the most economic and reliable welding process for bridge fabrication.

17. Key Words	18. Distribution Statement
Weldability, Bridge Welding Inspection, Codes & Specifications, Flaws, Metallurgical Aspects, Examples of Bridge Welding.	This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	201	

Form DOT F 1700.7 (8-72)

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METRIC CONVERSION FACTORS

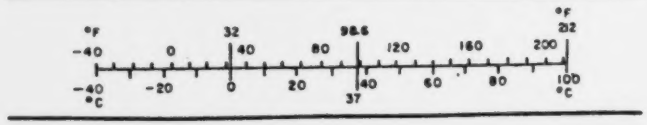
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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Section 1

INTRODUCTION

The advantage of welded bridge construction over bolted or riveted types consists in reduced weight and corrosion susceptibility (particularly at component junction points) as well as reduced construction space requirements and preparatory drilling and machining for anticipated fastening devices. On the other hand, weld preparation and planning become an important consideration. Also welding does not provide the inherent redundancy of a typical mechanically fastened bridge.

Welded bridges, because of their inherent lower cost to fabricate and their great asthetic appeal as compared to mechanically fastened bridges have come into wide spread use in recent years. Also, during recent years, a number of large scale fractures have been observed in these welded bridges which generally have not produced a catastrophic failure but have rather resulted in an unsafe condition which was located and corrected before any further harm could occur. The nature of these failures has brought about two thrusts in the bridge structural code. One is an attempt to better define the design of fracture critical bridge members and to insist upon much higher quality construction and inspection in these critical areas. The other thrust has been to look into welding technology and inspection technology to see what improvements might be made so as to provide inherently greater reliability of these welded connections from a standpoint of : 1) actual operating conditions, and 2) an assurance to regulatory agencies (Federal and State highway departments) that indeed these bridges have been constructed according to the codes and specifications as they exist at the time of fabrication (ideally while not raising fabrication costs).

The research program being undertaken by GARD is largely aimed in the latter direction, that is to look at means, both through mechanical and nondestructive testing, of assuring a higher quality weld joint in the bridge, thus greater operating reliability of the bridge. The program has been broken down into these two primary areas, one looking at improved mechanical testing methods for bridges and the other for improved nondestructive testing and quality control of bridges. The stated objectives of this contract are:

1. To establish the adequacy of current fracture toughness testing of weldments and to investigate new test methods for assessing toughness of welds.
2. To develop criteria to apply the acoustic emission(AE) technology to in-process and post weld monitoring to improve quality control and fabrication of welded structural elements.
3. To establish specific requirements for quality control and quality assurance to improve the quality and reliability of welded connections (design criteria, welding procedures, and fabrication techniques).
4. To establish specific requirements for quality control and quality assurance of weld repairs.
5. To prepare guidelines for the implementation of research results including a comprehensive discussion regarding the feasibility, applicability, cost effectiveness, and compatibility of the developed AE systems as applied to current fabrication practices and technology.

Prior to the actual construction and test phase of this program, the contract requires that a literature search be made to evaluate current fabrication procedures in bridge construction with respect to the following areas:

1. Weldability of steels and related tests.

2. Weld acceptance and qualification procedures, quality control and quality assurance as related to fabrication.
3. Weld repairs, their frequency, related quality and quality assurance procedures.
4. Inspection techniques employed for quality control and quality assurance.

Also, we were required in a separate task to evaluate the specific factors which result in weld flaws.

This report contains a summary of the integrated results for the two tasks: Literature Search and Isolation of Specific Factors Which Result in Weld Flaws. It was felt that the research in one area should not be disassociated with the research in the other area, thus both are placed together in a comprehensive discussion of bridge welding and testing technology. To this end, this report has been divided in the following sections: Section 2: Materials. Section 3: Specifications and Codes (with respect to both welding and quality assurance). Section 4: Defects and their Implications. Section 5: Research Trends in these areas. Section 6: Examples of Bridge Construction.

The initial section on materials discusses the nature of the three materials of primary concern for this program (and the materials that will actually be tested): ASTM A36, A588 A514. These are the most common materials in welded bridge construction. A36 is the most commonly used bridge material as it is the lowest cost material of all three and is satisfactory for most bridge applications. The A588 and A514 are special steels that are used in limited situations and form the minor proportion of bridge welding in the U.S. It is important to understand the basic characteristics of these materials prior to any discussion on welding or related

defects. This is why this section has been placed before the other sections in the report.

The next section discusses the specifications and codes required for bridge welding and inspection. It contains an overview of the current specifications and requirements for bridge welding along with pertinent comments and observations about the level of development of these specifications with respect to both current technology and the perceived needs of the public as administered through the federal and state highway departments. This section may be considered a critical review of these specifications and codes and is not intended to be merely a condensed version. The critical approach is particularly apparent in reviewing the strengths and weaknesses of the current code specified NDT methods. This is an important consideration in view of the major effort in this program which is to provide improved inspection of bridge weldments through application of in process acoustic emission monitoring.

Section 4 is a discussion of the significance of weld defects. It is divided into four sub-areas: classification of defects, welding imperfections common to bridge structural elements, consequences of weld defects, and the apparent frequency of these defects. The intent of this section is to evaluate typical defects found in welded sections of a bridge so as to provide a basis of information from which to select defects to be simulated and studied during the acoustic emission portion of this proposed program. The evaluation of defects will also serve as an aid in the selection of specific welding procedures to be simulated in the mechanical properties and acoustic emission testing to be performed in the main phase of this program.

Section 5 is concerned with recent research trends in the U.S. and foreign countries with respect to bridge welding, etc. The intent of this

section is to serve as a review of what is currently happening in bridge research so as to avoid duplication within this program. Emphasis has been placed on the review of material properties testing technology especially within a weldment. The major effort in this program is to improve the detection of defects and their classification. It is desirable also to have an evaluation of the potential growth and behavior of defects in a typical bridge environment because a bridge, as with any welded structure, is likely to have a number of defects that are either undetected or detected and left unrepaired.

This has been the focus of a great deal of research in the past twenty years of which the greatest effort has gone into the area of fracture mechanics analysis of defects, with a somewhat lesser effort spent in the study of the behavior of materials under fatigue. Research conducted during the 1940's and 50's on fracture toughness concentrated on the effect of notches in materials but did not really account for the effect of these notches in a realistic theoretical manner. This led to the development of linear elastic fracture mechanics and the associated experimental sciences of cyclic crack growth, corrosion induced crack growth, and most recently crack arrest. The object of the bridge designer is to find material properties which correlate well with bridge behavior in the real operating environment. Due to the newness of fracture mechanics as a science and its many undefined details, it is constantly necessary to reevaluate the state of the art in fracture mechanics prior to conducting any research programs in this area to ensure that the approaches taken are in keeping with the latest knowledge.

The final section of this report is concerned with several examples of bridge welding in foreign countries from a case study standpoint. The intent of these examples is to illustrate different approaches being taken outside

of the U.S. to resolve some of the bridge problems with respect to materials or welding techniques. Only a prolonged period of use can determine whether the approach taken in these examples will be successful or not successful, but the logic used in the selection of approach is illustrative of different approaches that may be taken to solve known bridge problems. The testing and analysis done by these foreign countries prior to construction of these bridges is of background interest to us prior to selection of acoustic emission parameters and specific types of the mechanical testing to be performed in this program.

The voluminous number of publications available made it necessary to concentrate on topics of more recent interest. Most sources originated through a literature search in computerized systems such as Metadex, Dialog, Compendex. Other information was obtained through direct contact with officials of specifying bodies such as FHWA, AWS, on a national level. On a state level, representatives were contacted from Illinois and New York State Departments of Transportation. Other agencies contacted were AASHTO and NCHRP.

The literature survey revealed that authorities on bridge welding construction are not centered at one particular institute or in one person but are represented by a multitude of different authors at different employers. Even major contributors to bridge welding did not publish more than 3 papers over the last 10 years.

Section 2

WELDABILITY OF STEELS USED FOR BRIDGE CONSTRUCTION

2.1 Introduction

Many metallurgical factors enter into the construction of a welded bridge. Among the most important is the selection of the basic structural materials. Three steels are most commonly used for bridge construction and are approved by regulatory agencies for the construction of welded bridges. These three materials fall into the classification of strong, low alloy steels. The American Society for Testing and Materials (ASTM) defines these steels as A 36, A 514, and A 588. These materials have the combination of strength and weldability needed for bridge welding applications. A summary of their key characteristics is given in Table 1.

To understand how these steels must be used in welded bridge construction, one must evaluate their chemical composition, manufacturing specifications, and mechanical properties. Other factors must also be taken into account such as, failure criteria (fatigue and fracture), hydrogen problems, corrosion, residual stresses, distortion and so on.

Base materials are only one part of the welding process. Consideration must also be given to filler metals and welding fluxes.

2.2 Base Materials

High strength structural steels with good welding properties first made their appearance in the early 1920's. Their unique properties depended on the new technique of controlled substitution of one element for another. In this case, manganese was substituted for carbon. This technique was used to give high strength structural steels weldability.

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TABLE 1 [1)]

TABLE 1 PROPERTY LEVELS AND USES FOR A 36, A 515, AND A 588 TYPE STEELS

ASTM specification	Title	Alloying elements(b)	Available mill forms	Special characteristics	Intended uses	Tensile strength MPa	Tensile strength ksi	Yield strength MPa	Yield strength ksi	Elongation in 200 mm or 8 in., %	Elongation in 50 mm or 2 in., %
A36	High strength Carbon Steel	Mn	Plates, any thickness	Standard Steel for welded Strucutres incl. Bridges, lowest cost per pound		400-550	58-80	220-250 (b)	32-36 (b)	20(b)	23
A514	High stength alloy steel	Cr, Ni Mo, Cu Zr, B V, Ti	Plates to 6 in.	Excellent Corrosion Resistance, 15 grades of similar strength	Primarily bridge weldments	690-895 (b)	100-130 (b)	620(b)	90(b)		15
A588	High-strength low-alloy structural steel with 50 ksi minimum yield point to 4 in. thick	Nb, V, Cr, Nl, Mo, Cu, Si, Ti, Zr.	Plates, bars and shapes up to 203mm (8 in.) thick.	Atmospheric corrosion resistance four time that of steel: nine grades of similar strength	Welded, bolted or riveted structures but primarily welded bridges and buildings where weight savings or added durability is important	435(b)	63(b)	290(b)	42(b)	16	21

(a) Where a single value is shown at is a minimum, (b) Minimum and or maximum values depend on the plate thickness.

Today, the current range of structural steels includes elastic limits over 50 ksi (345 MPa)[2] and carbon levels less than 0.2%. The most recent trends in manufacturing of these steels, have been in reducing the carbon and impurity levels. This practice allows the deliberate addition of micro-alloying elements such as B, V, Ti, and Nb.[3] These elements are added to enhance, or optimize the weldability of these structural steels, while retaining the strength and toughness that are needed for construction. At times these additions are coupled with thermal, and/or thermal-mechanical treatments.

These additions of micro-alloying elements occur in small amounts. For example, addition of Nb is in the range of 0.01-0.05%, and V in the range of 0.05-0.1%. The additions act through physical changes in the microstructure, such as solid-solution strengthening, grain refinement, and precipitation strengthening. The addition of these elements also retains strength and toughness at lower temperatures, and increases the weldability at elevated temperatures (see Table 2). The increased weldability is a result of alloy controlled precipitation of carbonitrides, which affects dislocation movement and subsequent grain growth.[3] These developments have made the welding of bridges a more attractive alternative to other bridge construction techniques.

2.2.1 A36 Steel

This material is considered a plain carbon steel with intermediate strength. The carbon level for A36 steel falls in the range of 0.2-0.3%. Table 3 illustrates the ASTM requirements for chemistry, and mechanical properties of this steel. Manganese is intentionally added to about 1%, in order to increase its basic strength without raising the carbon level beyond a level where weldability is impaired. When the manganese level in A 36 steel exceeds 1%, chromium is added in place of manganese. This is done because manganese and chromium are interchangeable, and when the manganese levels exceed 1.5%, the toughness of the steel is reduced.

TABLE 2

SUMMARY OF THE EFFECTS OF MICROALLOYING ADDITIONS ON WELDABILITY (a)

MICROALLOYING ADDITION	ASPECT OF WELDABILITY	
	HYDROGEN CRACKING	SOLIDIFICATION CRACKING
Vanadium	Reductions in carbon content can offset the increased risk of cracking. Data available	Probably beneficial
Columbium	No effect to about 0.05%. No data for higher levels	Detrimental at least to about 0.13%. Less effect at lower levels.
Titanium	No data.	Limited, conflicting data.
Zirconium	No data.	No data.
Boron	Increases the risk of cracking, but can be offset by reductions in carbon content.	Probably detrimental but could be offset by reductions in carbon content.

(a) Constant base composition.

TABLE 2

SUMMARY OF THE EFFECTS OF MICROALLOYING ADDITIONS ON WELDABILITY [a] (Continued)

MICROALLOYING ADDITION	ASPECT OF WELDABILITY	
	HEAT AFFECTED ZONE TOUGHNESS	WELD METAL TOUGHNESS
Vanadium	Little effect at low levels, less than about 0.08%. At higher levels generally detrimental. Tolerable amount increased by lower carbon content	Complex pattern because of grain refining and precipitation hardening effects. Some quantitative data available.
Columbium	Little effect at low inputs, detrimental at high heat inputs to about 0.06%. No data for higher levels.	Complex pattern because of grain refining and precipitation hardening effects.
Titanium	No data except for very high heat input welds where beneficial effect of titanium nitride on grain growth can be exploited.	Complex pattern because of grain refining and precipitation hardening effects. Some quantitative data available.
Zirconium	No data.	Mainly detrimental according to limited data.
Boron	No data.	Good and bad effects dependent on circumstances.

(a) Constant base composition.

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TABLE 3

A36 CARBON STEEL - CHEMICAL COMPOSITION AND REQUIREMENTS, SPECIFICATIONS, AND MECHANICAL PROPERTIES(31)

PRODUCT	(a) SHAPES	PLATES					BARS			
Thickness in. (mm.)	ALL	To 3/4 (19) incl.	Over 3/4 to 1 1/2 (19 to 38) incl.	Over 1 1/2 to 2 1/2 (38 to 64) incl.	Over 2 1/2 to 4 (64 to 102) incl.	Over 4 (102)	To 1/4 (19) incl.	Over 3/4 to 1 1/2 (19 to 38) incl.	Over 1 1/2 to 4 (102) incl.	Over 4 (102)
Carbon max. %	0.25	0.25	0.25	0.26	0.27	0.29	0.26	0.27	0.28	0.29
Manganese	—	—	0.80 to 1.20	0.80 to 1.20	0.85 to 1.20	0.85 to 1.20	—	0.60 to 0.90	0.60 to 0.90	0.60 to 0.90
Phosphorus max.%	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
Sulfur max.%	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
Silicon %	—	—	—	0.15 to 0.30	0.15 to 0.30	0.15 to 0.30	—	—	—	—
Copper min.% when Cu steel is Specified	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
Tensile Strength	58,000 to 80,000 psi (400 to 550 MPa)									
Yield Point min.	36,000 psi (250 MPa) (d)									
Elongation in 8 in. or 200 mm. min.%	20(c)	20(b)	20(b)	20(b)	20(b)	20(b)	20	20	20	20
Elongation in 2 in. or 50 mm min. %	21(c)	23	23	23	23	23	23	23	23	23

(a) Manganese content of 0.85 1.35% and silicon content of 0.15 0.30% is required for shapes over 426 lb/ft.

(b) For plates wider than 24 in. (610 mm), the test specimen is taken in the transverse direction.

(c) For wide flange shapes over 426 lb/ft. tensile strength minimum of 58,000 psi (400 MPa) only and elongation in 2 in. of 19% minimum applies.

(d) Yield point 32,000 psi (220 MPa) for plates over 8 in. in thickness.

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2.2.2 A514 Steel

For applications where high strength is required, A514 steel is a common replacement for A36 type materials. This material readily hardens using normal quench and temper procedures. This feature distinguishes A514 steel from A36 steel compositions. This is accomplished by alloying with chrome and molybdenum. Boron is also added to keep the chrome and molybdenum levels low. Boron is used to reduce alloying costs, chrome and molybdenum being very expensive, and to keep the steel in the low alloy steel group. Boron has the same effect as chrome and molybdenum has on steel. Generally A514 is referred to as alloy plate to distinguish it from the lean A36 steel it replaces in critical locations. Table 4 illustrates the ASTM requirements for chemistry and mechanical properties for this group of steels.

Other differences set A514 steels apart from A36 steels. In A514 steel, the carbon level is allowed to drop into the range between 0.1 and 0.2%, making this steel weldable under adverse conditions. The strength is achieved by chromium additions balanced by a specified amount of molybdenum to prevent temper embrittlement in anticipation of the thermal effects of welding. While the A514 type steels display high yield strength and good resistance to brittle fracture, welding has a critical effect on performance when it is not done properly. Boron is now used primarily to balance the through section properties in heavier section sizes.

Fracture toughness (K_{IC}), Charpy V-notch (CVN), and Dynamic Tear Test (DT) results for A514 steels are shown in Figure 1*, indicating the degrees to which temperature, loading rate, and thickness affect these values. [4)]

Recently low and high manganese content steels have been developed that

* When there is more than one figure, table or a combination of figures and tables listed on one text page they will follow immediately after that page in the same sequence as they are found in the text, except when specified on that particular figure or table.

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TABLE 4

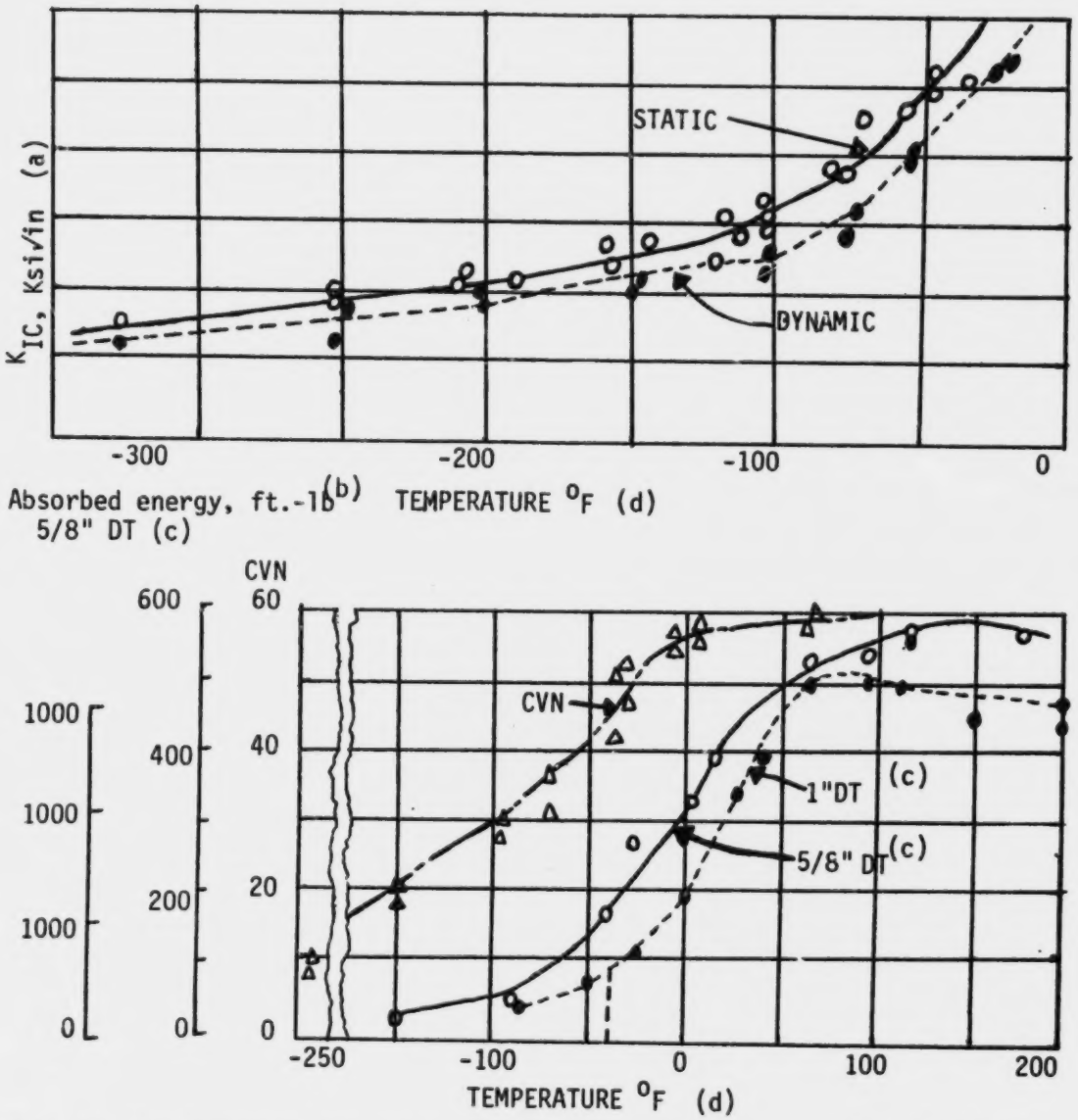
A514 CARBON STEEL, CHEMICAL COMPOSITION AND REQUIREMENTS, SPECIFICATIONS, AND MECHANICAL PROPERTIES(31)

A514, GRADE B Used by GARD for experimental bridge welding	Thickness maximum in. (mm)	Carbon %	Manganese %	Phosphorus max. %	Sulfur max. %	Silicon %	Chromium %	Molybdenum %	Vanadium %	Titanium %	Boron %
	1 1/4 (32.0mm)	0.12 to 0.21	0.70 to 1.00	.035	0.04	0.20 to 0.35	0.40 to 0.65	0.15 to 0.25	0.03 to 0.08	0.01 to 0.03	0.005 to 0.005

For a complete listing of all 514 steel grades (types) consult the Metals Handbook Ninth Edition, Volume 1 - Properties and Selection: Iron and Steels.

THICKNESS INCHES	Ultimate Tensile Strength Ksi (MPa)	Yield Strength minimum, ksi(MPa) (a)	Elongation in 2 in. or 50 mm min.% (b & c)	Reduction of Area min % (c)	Brinell Hardness Number
To 3/4 incl.	110 (760) to 130 (895)	100 (690)	18	40(e)	235 to 293
Over 3/4 to 2 1/2 incl.	110 (760) to 130 (895)	100 (690)	18	40(e) 50(f)	235 to 293
Over 2 1/2 to 4 incl.	100 (690) to 130 (895)	90 (620)	17	50 (f)	229 to 293

(a) Measured at 0.2% offset or 0.5% extension under load, yield strength of ASTM Methods and Definitions A370 for Mechanical Testing of Steel Products.
(b) Elongation not required to be determined for floor plate.
(c) For plates wider than 24 in. (610 mm), the transverse elongation is reduced two percentage points and the transverse reduction of area is reduced five percentage points.
(d) Either the full thickness rectangular specimen, Methods A370 (see footnote b above), or the 1/2 in.(12.5mm) diameter specimen, may be used for plates over 3/4 in. to 1 1/2 in.(19 to 40mm) in thickness.
(e) Measured on 1 1/2 in.(40mm) wide full thickness rectangular specimen, Methods A370. This is the only specimen to be used for thicknesses of 3/4 in.(19mm) and under. The elongation is measured in a 2 in or 50 mm gage length which includes the fracture and which shows the greastest elongation.
(f) Measured on 1/2 in. (12.5mm) diameter specimen, Methods A370. This is the only specimen to be used for thicknesses over 1 1/2 in. (40mm). The specimen shall be taken so that the axis is midway, or as near midway as practicable, between the center and the surface.



(a) To obtain equivalent units in MPa√cm multiply above values by 10.97.
(b) To obtain equivalent units in Joules multiply above values by 1.36.
(c) To obtain equivalent units in cm multiply above values by 2.54.
(d) To obtain equivalent units in °C subtract 32.0, and multiply by .56 the corresponding above values.

Figure 1 FRACTURE TEST RESULTS ON A514 STEEL SHOW THAT NOTCH TOUGHNESS VARIES WITH PLATE THICKNESS AS WELL AS TEST TEMPERATURE.(31)

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are similar to A514 steels. Delvecchio et.al.[5] have shown that these steels have the potential of meeting 70 RC strength and toughness specifications. These materials would be ideal for welding bridge construction. At the .2%Mo. level, the higher manganese content steels are better suited because they can achieve consistently higher yield strength levels than the low-manganese steels. These steels are attractive from a standpoint of tolerance to low temperatures, rolling requirements, and higher production rates during the controlled rolling sequence. However, as expected, increasing the alloy and carbon contents in the steels raised the underbead hardness, thus increasing the susceptibility to hydrogen induced cracking, Figure 2. This indicates that both the carbon and manganese contents should be controlled to meet mechanical property specifications while maintaining field weldability. The high manganese steel can not be used due to the increased hydrogen cracking, and the low manganese steel can not be used because of the reduced weldability. Type A514 is still the best of the three materials to use for welding bridges.

2.2.3 A588 Steel

A588 has proven to be a popular alternate to A514 type steels, when improved weldability and atmospheric corrosion resistance are concerned, (Table 5). An example of the utilization of this superior necessary resistance is in the manufacturing of preassembled trade units that have more girders and more uniform construction, than trade units now in use. These units are replacing corroded railway bridge sections, increasing their design strength and corrosion resistance.

A588 steel is considered a high strength-low alloy steel, although its strength level falls half way between the levels for A36 and A514, and its alloy content is at about the same level as for A514 steels. However, vanadium and nickel have been substituted for molybdenum and copper.

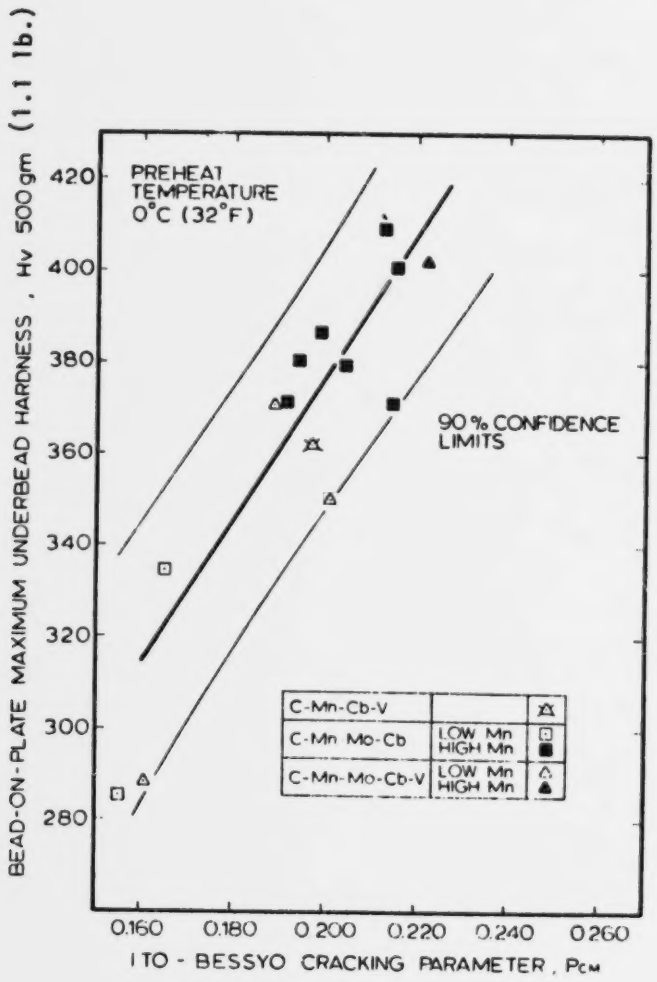

Figure 2 BEAD-ON-PLATE UNDERBEAD HARDNESS VERSUS P_{CM} (31)

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TABLE 5

CORROSION OF STRUCTURAL STEELS IN VARIOUS ENVIRONMENTS

(C. P. Larrabee and S. K. Coburn, "The Atomospheric Corrosion of Steels as Influenced by Changes in Chemical Composition", Metallic Corrosion - First International Congress on Corrosion, Butterworth's, London, 1962, pp. 276-284)

Type of atmosphere	Time, yr	Average reduction in thickness, mils (a): Structural carbon steel	Structural copper steel	UNS K11510(b)	UNS K11430(c)	UNS K11630(d)	UNS K11576(e)
Industrial (Newark, NJ)	3.5	3.3	2.6	1.3	1.8	1.4	2.2
	7.5	4.1	3.2	1.5	2.1	1.7	...
	15.5	5.3	4.0	1.8	...	2.1	...
Semi-Industrial (Monroeville, PA)	1.5	2.2	1.7	1.1	1.4	1.2	1.6
	3.5	3.7	2.5	1.2	2.1	1.4	2.4
	7.5	5.1	3.2	1.4	2.4	1.7	...
	15.5	7.3	4.7	1.8	...	1.8	...
Semi-Industrial (South Bend, PA)	1.5	1.8	1.4	1.0	1.3	1.0	1.5
	3.5	2.9	2.2	1.3	1.9	1.5	2.4
	7.5	4.6	3.2	1.8	2.7	1.9	...
	15.5	7.0	4.8	2.2	...	2.5	...
Rural (Potter County, PA)	2.5	...	1.3	0.8	1.2	...	...
	3.5	2.0	1.7	1.1	1.4	1.2	1.8
	7.5	3.0	2.5	1.3	1.5	1.5	...
	15.5	4.7	3.8	1.4	...	2.0	...
Moderate Marine (Kure Beach, NC, 800 ft. (244m) from ocean	0.5	0.9	0.8	0.6	0.8	0.7	1.0
	1.5	2.3	1.9	1.1	1.7	1.2	1.7
	3.5	4.9	3.3	1.8	2.5	1.9	2.2
	7.5	5.6	4.5	2.5	3.7	2.9	...
Severe Marine (Kure Beach, NC, 80 ft. (24m) from ocean	0.5	7.2	4.3	2.2	3.8	1.1	0.7
	2.0	36.0	19.0	3.3	12.2	...	2.1
	3.5	57.0	38.0	...	28.7	3.9	3.9
	5.0	(f)	(f)	19.4	38.8	5.0	...

(a) To obtain equivalent values in mm, multiply listed value by 25 and to obtain equivalent values in inches multiply listed values by 0.04.

(b) ASTM A242 (Type 1).

(c) ASTM A588 (Grade A).

(d) ASTM A514 (Type B) and A517 (Grade B).

(e) ASTM A514 (Type F) and A517 (Grade F).

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Zirconium is added in balanced amounts for desired through section properties, and atmospheric corrosion resistance. Steels with very small columbium or vanadium levels are also referred to as micro-alloyed steels.

There are some restrictions that must be observed when using A588 steel. A588 steels are vulnerable to fatigue cracking at flame cut, or tack weld locations. The New York State Department of Transportation (NYSDOT);[6] requires heat treatment to bring the edges exposed to tensile stresses within a 30 RC hardness value to prevent fatigue cracking. Subsequent welding is to be performed with specified heat input depending on thickness of the steel. This is done because the affected zone is limited to a hardness value of 27 RC. The preheat and interpass temperatures must not fall below 150^{o}F (65.5^{o}C) in field welding. No heat curing is allowed.

Table 6 illustrates the ASTM requirements for A588 steel chemical composition ranges, and minimal mechanical properties. Table 7 lists domestic producers, and commercial designations for equivalent type A588 steels. As seen, the material is readily available although significant variations in available carbon levels occur from supplier to supplier for a particular grade.

2.3 Welding Wire

Various steels, both carbon and alloy, are used to produce welding wire for the manufacture of electrodes and rods for electric and gas welding. While some gas welding is still employed, by far the greater proportion of welding operations are performed by the electric-arc process. For the fabrication of bridges gas welding is prohibited. The development of the present-day HSLA steels has paralleled the growth of the various welding processes, and

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TABLE 6

A588 CARBON STEEL - CHEMICAL COMPOSITION AND REQUIREMENTS, SPECIFICATIONS, AND MECHANICAL PROPERTIES (31)

A588: GRADE A, USED BY GARD FOR EXPERIMENTAL BRIDGE WELDING - CHEMICAL REQUIREMENTS								
ELEMENT	CARBON	MANGANESE	PHOSPHORUS	SULFUR	SILICON	CHROMIUM	COPPER	VANADIUM
COMPOSITION %	0.10 to 0.19	0.90 to 1.25	0.04 max.	0.05 max.	0.15 to 0.30	0.40 to 0.65	0.25 to 0.40	0.02 to 0.10
For a complete listing of all A588 steel grades (types) consult the Metals Handbook Ninth Edition, Volume 1 - Properties and Selection: Iron and Steels								

TENSILE REQUIREMENTS (a)				
	PLATES and BARS			STRUCTURAL SHAPES
	For thicknesses 4 in and under (101.6mm)	For thicknesses over 4 in to 5 in. incl.(101.6 to 127.0mm)	For thicknesses over 5 in.to 8in. incl.(127.0 to 203.2mm)	ALL GROUPS
TENSILE STRENGTH, MIN,psi (MPa)	70,000 (485)	67,000 (460)	63,000 (435)	70,000 (485)
Yield POINT, MIN., psi (MPa)	50,000 (345)	46,000 (315)	42,000 (290)	50,000 (345)
ELONGATION IN 8 in. OR 200mm, MIN. %	18 (b,c)	—	—	18
ELONGATION IN.2 in. OR 50mm, MIN.,%	21 (b)	21 (b)	21 (b)	21 (d)

(a) For plates wider than 24 in. (160mm), the test specimen is taken in the transverse direction.
(b) Elongation not required to be determined for floor plate.
(c) For plates wider than 24 in. (610mm), the elongation requirement is reduced two percentage points.
(d) For wide flange shapes over 426 lb/ft elongation in 2 in. of 18% minimum applies.

TABLE 7

Domestic Producers and their Specifications for Various A588 Type Steels. (42)

NAME	(a) PRODUCER	THICK-NESS INCHES (mm)	COMPOSITION PERCENT (b)											MECHANICAL PROPERITIES(c)		ATMOS-PHERIC CORROSION RESISTANCE TIMES C STEEL
			C	Mn	P	S	Si	Cu Min.	Cr	Ni	Mo	V	OTHER	YIELD STRENGTH 100 psi (MPa)	ULTIMATE TENSILE STRENGTH 1000psi (MPa)	
ARMCO HIGH-STRENGTH A588	ARM	TO 4 (102)	0.20	0.75 to 1.25	0.04	0.05	0.15 to 0.30	0.20 to 0.40	0.40 to 0.70	0.25 to 0.50	—	0.01 to 0.10	—	50(345)	70 (485)	4
MAYARI R-50	BETH	TO 4 (102)	0.20	0.75 to 1.25	0.04	0.05	0.15 to 0.30	0.20 to 0.40	0.40 to 0.70	0.25 to 0.50	—	0.01 to 0.10	—	50(345)	70 (485)	4
MAYARI R-60	BETH	TO 1 (25.4)	0.20	0.75 to 1.35	0.04	0.05	0.15 to 0.30	0.20 to 0.40	0.40 to 0.70	0.25 to 0.50	—	0.01 to 0.10	—	60(415)	80 (550)	4
Ni-Cu-Ti	JL	—	0.15	1.00	0.04	0.05	0.50	0.30	—	0.70	—	—	0.05	50(345)	70(485)	4
KAISALOY 45 FG	K	TO 1/2 (13)	0.12	0.60	0.35	0.04	0.50	0.30	0.25	0.60	0.10	0.02 Min	Ti .005	45(310)	60 (415)	UP TO 6
KAISALOY 50 CR	K	TO 4 (102)	0.20	1.25	0.35	0.04	0.25 to 0.75	0.20 to 0.35	0.10 to 0.25	0.30 to 0.60	0.15	0.02 to 0.10	Ti .005 to 0.03	50(345)	70(485)	5 to 8
LUKENS A588 Grade A	LUK	TO 4 (102)	0.10 to 0.19	0.90 to 1.25	0.04	0.05	0.15 to 0.30	0.25 to 0.40	0.40 to 0.65	—	—	0.02 to 0.10	—	50(345)	70(485)	> 4
A588 GradeA LUK-MOD-3	LUK	TO 1 1/2 (38)	0.25	1.50	0.04	0.05	0.15 to 0.30	0.25 to 0.40	0.40 to 0.65	—	—	0.02 to 0.10	—	60(415)	77(535)	> 4
N-A-X High TENSILE	NAT	—	0.18	0.50 to 0.90	0.04	0.04	0.60 to 0.90	—	0.40 to 0.80	—	—	—	Zr 0.03 to 0.12	50(345)	70(485)	4 to 6
N-A-X 80	NAT	—	0.12	1.00	0.02	0.03	0.90	0.80	—	—	—	—	Cb.01 Zr.06	80(590)	90(620)	4 to 6

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TABLE 7

DOMESTIC PRODUCERS AND THEIR SPECIFICATIONS FOR VARIOUS A588 TYPE STEELS [42] (Continued)

ORELLOY 588	ORE	TO 2 (51)	0.19	1.25	0.04	0.05	0.30	0.25 to 0.40	0.40 to 0.65	—	—	0.02 min.	—	50(345)	70(485)	4 to 6
REPUBLIC 50	REP	—	0.15	0.50 to 1.00	0.04	0.05	—	0.30 to 1.00	0.30	0.40 to 1.10	0.10 min.	0.02 min.	Cb 0.01 min.	50(345)	70(485)	4 to 6
REPUBLIC 60	REP	—	0.15	0.50 to 1.00	0.04	0.05	—	0.30 to 1.00	0.30	0.40 to 1.10	0.10 min.	0.02 min.	Cb 0.01 min.	60(415)	80(550)	4 to 6
COR-TEN B	USS [d]	TO 4 (102)	0.10 to 0.19	0.90 to 1.25	0.04	0.05	0.15 to 0.30	0.25 to 0.40	0.40 to 0.65	0.65	—	0.02 to 0.10	A1	50(345)	70(485)	>4
YOLOY HS	YST	TO 1/2 (13)	0.15	1.00	0.04	0.05	0.30	0.50 to 1.00	—	0.75 to 1.25	0.10 to 0.25	—	—	50(345)	70(485)	4 to 6
YOLOY HSX	YST	> 1/2 (13)	0.15	1.25	0.04	0.05	0.30	0.50 to 1.00	—	0.75 to 1.25	0.10 to 0.25	—	—	45(310)	65(450)	4 to 6
ALGOTUF 50	ALG	PLATE	0.18	1.25	0.03	0.05	0.30	0.20 to 0.40	0.60	0.50	—	0.02 min.	—	50(345)	70(485)	4 to 5
ALGOTUF 70	ALG	PLATE	0.20	1.60	.025	.035	0.35	0.50	—	0.50	—	0.10 min.	Cb 0.06 min.	70(485)	90(620)	4 to 6
ALGOMA 50A	ALG	—	0.20	0.75 to 1.35	0.03	0.04	0.15 to 0.40	0.20 to 0.60	0.70	(e) 0.90	—	—	Cb + v 0.10	50(345)	70/95 (485/655)	>4
ALGOMA 60A	ALG	—	0.20	0.75 to 1.35	0.03	0.04	0.15 to 0.40	0.20 to 0.60	0.70	(e) 0.90	—	—	Cb +v 0.10	60(415)	75/100 (520/690)	>4
ALGOTUF 50	ALG	SHEET	0.18	1.25	0.03	0.05	0.30	0.20 to 0.40	0.60	0.50	—	0.02 min.	—	50(345)	70(485)	4 to 5
ALGOTUF 70	ALG	SHEET	0.20	1.60	.025	.035	0.35	0.50	—	0.50	—	0.10 min	Cb 0.06 min	70(485)	90(620)	4 to 6

TABLE 7

DOMESTIC PRODUCERS AND THEIR SPECIFICATIONS FOR VARIOUS A588 TYPE STEELS [42] (Continued)

STELCOLOY 50	STE	TO 4 (102)	0.15	1.35	0.03	0.04	0.15 to 0.30	0.20 to 0.50	0.30 to 0.50	0.25 to 0.50	–	0.01 min	–	50(345)	70(485)	4 to 5
STELCOLOY 60	STE	TO 1 1/2 (38)	0.20	1.00 to 1.50	0.04	0.04	0.15 to 0.30	0.20 to 0.50	0.50	0.25 to 0.50	–	0.01 min.	–	60(415)	80(550)	4 to 5
STELCOLOY 70	STE	TO 3/4 (19)	0.22	1.50	0.03	0.04	0.15 to 0.30	0.20 to 0.50	–	0.25 to 0.50	–	0.02 to 0.10	Cb 0.10	70(485)	85(585)	2 to 4
STELCO A588 GRADE C	STE	TO 4 (102)	0.15	0.80 to 1.35	0.04	0.05	0.15 to 0.30	0.20 to 0.50	0.30 to 0.50	0.25 to 0.50	–	0.01 to 0.10	–	50(345)	70(485)	>2

(a) Code for producers; U. S. Producers: ARM=Armco Steel Corporation; BETH=Bethlehem Steel Corporation; JL=Jones and Laughin Steel Corporation; K= Kaiser Steel Corporation; LUK= Lukens Steel Company; NAT= National Steel Corporation; ORE= Oregon Steel Mills, Division of Gilmore Steel Corporation; REP= Republic Steel Corporation; USS= United States Steel Corporation; YST= Youngstown Sheet and Tube Company; Canadian Producers: ALG= The Algoma Steel Corporation; STE= The Steel Company of Canada, Ltd. (Stelco).
Maximum values are listed except where ranges or minimum values are indicated or where parentheses are used to denote typical values.
Minimum values are given unless parentheses are used to denote typical values.
Atmospheric corrosion resistance can be improved to about twice that of plain carbon steel (without Cu) by adding at least 0.20% Cu (optional).
Controlled rolled in thicknesses up to 3/4 inch (19mm) for certified notch toughness
0.005-0.05 Cb and/or 0.01-0.10V, or 0.01-0.10V+0.015 N maximum
Increased to 70,000 psi (485 MPa) minimum after prestrain and an aging treatment.

(b) In some cases V, Zr, or Ti may be used in these steels.
(c) 70 Ksi for ASTM A606.
(d) These elements may be used singly or in combination so long as the total does not exceed 0.10 percent.
(e) Normalized for certified notch toughness; Grade C meets 25 ft-lb (34J) at -50°F (-45°C) (longitudinal) through 2 inches in thickness.

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particular care was exercised to make certain that these steels possessed suitable welding characteristics. Most of these steels are considered to be readily weldable by conventional processes.

For shielded-metal-arc welding of HSLA steels having minimum yield points up to about 50,000 psi (345 MPa), E60 or E70 group mild-steel covered electrodes are generally satisfactory. E70 grade electrodes are suggested for steel grades having somewhat higher minimum yield points. For heavier sections and for grades that have higher carbon and manganese contents, pre-heating and/or low-hydrogen-type electrodes such as E7016 or E7018 are required. Low-alloy-steel electrodes are generally required for steels with minimum yield points higher than about 60,000 psi (414 MPa) or when specific characteristics, such as enhanced corrosion resistance, are required in the weld metal. Electrodes or electrode-flux combinations that provide filler metal similar to that of the suggested electrodes for shielded-metal-arc welding are recommended for submerged-arc, gas-metal-arc and flux-cored-arc welding.

Electric-arc welding is divided into two classifications: manual welding and automatic welding.

Wire for manual welding is furnished in straightened and cut lengths with surface as free from wire-drawing lubricant as possible to enable electrode manufacturers to apply a flux coating by an extrusion method. Welding steels are made especially for this application, and selection of heats or portions of heats is mandatory to conform to required composition limits and ranges. Low sulphur, not exceeding 0.035 per cent, is generally preferred. Most of the grades of steel made for welding electrodes are of the rimming type, with carbon in the neighborhood of 0.10 to 0.15 per cent. However, there is also some demand for rimming-type steel with 0.31 per cent carbon as the maximum of the range, but this carbon level is not recommended for bridge welding.

Some alloy grades are used in the manufacture of electrodes for special purposes such as resistance to abrasion. Composition of the steel is governed by composition of the flux coating, the combination being designed to provide certain welding characteristics and specific properties in the deposited metal.

The wire thus serves as the core of the coated electrode and is expected to provide good welding characteristics and a dense, sound deposit. Wire sizes in popular demand are 1/8-inch (.32cm), 5/32-inch (.40cm), 3/16-inch (.48cm), and 1/4-inch (.64cm). Tolerances for diameter, length, camber, and burrs in the straightened and cut wire are listed in the AISI Products Manual for Wire.

The flux coating on the electrode provides a blanket which surrounds the arc during welding and serves as a protection against oxidation from the atmosphere. In automatic electric-arc welding, there are two main processes: submerged-arc and inert-gas. In the former, the welding is shielded by a blanket of fusible material, usually referred to as flux which is fed on to the work. In the latter, shielding is obtained from inert gas such as argon or helium. Both automatic and semi-automatic equipment is used for each process.

Wire compositions vary for submerged-arc welding according to requirements in the deposit metal. The wire is produced with a bright copper finish to improve the contact surfaces and so facilitate the introduction of the relatively high currents used. It is important that the surface of the wire be as free as possible from wire-drawing lubricant which would tend to clog the nozzle through which the wire is fed. Wire for submerged-arc welding is furnished in layer-wound coils with cardboard cores. Dimensions of these coils are designed for insertion into a reel from which the wire is fed through the welding machine. Wire sizes in demand range from about 3/32-inch(.24cm) up to about 5/16-inch (.79cm).

The inert-gas welding process calls for smaller wires ranging from 0.045-inch (.11cm) to around 0.125-inch (.32cm). Wire for this application is furnished with a bright, clean copper coating for the same reason as mentioned above and is wound on spools of the dimensions necessary for insertion in the welding equipment. The required wire compositions vary according to requirements of consumers.

Section 3

BRIDGE WELDING AND INSPECTION AS GOVERNED BY CODES AND SPECIFICATIONS

3.1 Governing Codes and Specifications

Steel bridges could constitute a substantial hazard if catastrophic failures occur. Therefore, codes and standard specifications have been prepared to cover requirements for bridge design and construction. Those strict requirements are the result of many years of shop and field experience and continuous research in order to keep up with technical developments in this field.

Highway bridges in all fifty states are designed according to the Standard Specifications for Highway Bridges (SSHB), as adopted by the American Association of State Highway Officials (AASHO) which since 1973 has changed their name to the American Association of State Highway and Transportation Officials (AASHTO). The latest edition of the SSHB is the 12th edition of 1977[7]. In addition, three yearly revisions of the Interim Specifications (IS), bearing the year of their issue were published in 1978[8], 1979[9] and 1980[10], respectively.

According to the SSHB, welding of steel bridge structures conforms to the Structural Welding Code, prepared and published by the American Welding Society. The latest edition of the Structural Welding Code (AWS D1.1-80) was issued in 1980,[11] and covers general welding requirements applicable to any type of welded structures. However, in order to meet the specific needs of bridge construction this code must be used in conjunction with the Standard Specifications for Welding Structural Steel Highway Bridges (second edition, 1977)[12], and Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members (1978)[13] as adopted by AASHTO which are considered as a modification to the structural Welding Code AWS D1.1-75.

3.2 Base Metals

The Code and Specifications mentioned above provide various structural steels for plate and/or shapes, used specifically for bridge construction. These base metals are given in Table 8.

The steels listed in Table 8 are used for plates and/or shape members. Only one of them is carbon steel (A36), the rest are high strength low alloy (HSLA) steels. Two of them (A588, A242), provide atmospheric corrosion resistance ranging from 4 to 8 times that of carbon steel without copper additions and can be used in the unpainted condition.

Steels conforming to ASTM 514 are used when the deadweight reduction achieved through use of high yield strength material provides an economic advantage.

The detailed description and characterization of three of the most common steels used in welded bridge construction (A36, A588, and A514) is given in Section 2.

3.3 Design of Welded Connections

3.3.1 Joint Qualification

The AWS Structural Welding Code [11] has established certain joints as being prequalified because over a period of years these joints, used with conventional procedures have established a record of satisfactory service performance. This was proved by experience and numerous tests which preclude the need for further testing. These tests have shown that using filler metals conforming to a given AWS classification and meeting other requirements allows different manufacturers to produce welds which have similar properties.

That is why, unless otherwise instructed, the prequalified joints may be used without performing joint welding procedures qualification tests, provided:

TABLE 8

BASE METALS FOR BRIDGES

ASTM Designation	Grade	Specifications for	Steels prequalified by AASHTO Code or Speci. (7, 12)
A36		Structural Steel	x
A441		High-Strength Low-Alloy Manganese-Vanadium Steel	x
A514		High-Yield Strength, Quenched and Tempered Alloy Steel Plate	x
A517		High-Strength Alloy Steel Plates, Quenched and Tempered, for Pressure Vessels	x
A572	42	High-Strength Low-Alloy Columbium Vanadium Steels of Structural Quality	x
	45		X
	50		x
A588		High-Strength Low-Alloy Structural Steel	x
A242		High-Strength Low-Alloy Steel Plate	x*

Note: * Considered for use in bridge welding if its weldability investigation is conducted by special longitudinal-Bead-Weld Underbead Cracking Test.(12)

- o The base metal is recognized by appropriate codes or specifications.
- o The welding process is selected from the list of the prequalified ones.
- o Preheat and interpass temperature (and heat input control for quenched and tempered steel) are held to at least a specified minimum.
- o Joints are welded in a position to which their prequalification is restricted.
- o The AWS Code workmanship and technique are employed.

However, for fracture critical members or member components (FCMs)[13] which are tension members or tension components of members whose failure would be expected to result in collapse of the bridge, weld joint procedures are not prequalified, even if weld joint details comforming in all respects to the provisions of Section 2 of AWS D1.1 are used. The provision for qualification of the joint welding procedures for FCMs are outlined in Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members.[13]

Each detail for a welded joint, when not precisely defined by the specification should be selected after considering the basic service requirements, including the character of the applied load, the kind and thickness of steel, the welding process, the fabrication and erection method, etc.

There are three types of prequalified welds specified in the AWS Code:

1) fillet weld
2) complete joint penetration groove weld
3) partial joint penetration groove weld which is a weld performed from one side without steel backing, or a weld made from both sides but without back gouging.

Designers should specify the type of joint required. For fillet welds, the size, spacing, and other details of the welds should be clearly shown. The groove depth should be shown for partial joint penetration groove welds also.

For complete joint penetration groove welds, the designer should let the fabricator select and use the AWS prequalified joint, or the fabricator

qualified joint details best suited, unless he has a specific reason for showing a particular joint preparation. The joint selected should be reviewed and approved by the designer.

3.3.2 Fatigue Aspect in Design and Details of Welds

In most situations bridges are subject to dynamic and repetitive loads which constitutes the principal difference, other than geometry in the structural aspects of bridges. Other structures are subjected to static loads like, for example, buildings. This requires the bridge designer to consider additional material properties and design constraints that are beyond those required for building design.

In terms of fatigue the bridge members and fasteners subjected to repeated variations or reversals of stress are designed so that the actual range of stresses does not exceed the allowable fatigue stress range (algebraic difference between the maximum and minimum stress). These ranges for the appropriate type and location of material are classified as fatigue stress categories and designated by capital letters from A through E. The A-category allows the highest allowable range of stresses, while the E-category is restricted to the lowest range. Nonredundant load path members, (that is where failure of a single element could cause collapse) are usually specified for a lower range of stress than redundant ones of the same stress category.

For example, only a base metal with rolled and clean surfaces, and flame cut edges which have the smoothness specified by American National Standards Institute are viewed as A-category. However, the same base metal in members connected by continuous groove welds parallel to the direction of applied stress, as well as the weld metal itself are referred to the lower B-category.

The allowable fatigue stress classification shows that full penetration groove welds and adjacent base metal are designed for higher allowable stress ranges (higher category) than partial penetration welds. These welds are also

higher than fillet welds because the latter are similar to partial penetration welds in that they both have unwelded roots. This is why tension or compression members like splices in beams and girders subjected to dynamic load are made with complete penetration groove welds, the stress category being higher if the reinforcements are ground off.

Intermittent welds are considered the worst in terms of fatigue resistance and are not permitted in groove welds. When utilized in fillet welds, they are qualified as the lowest E-category.

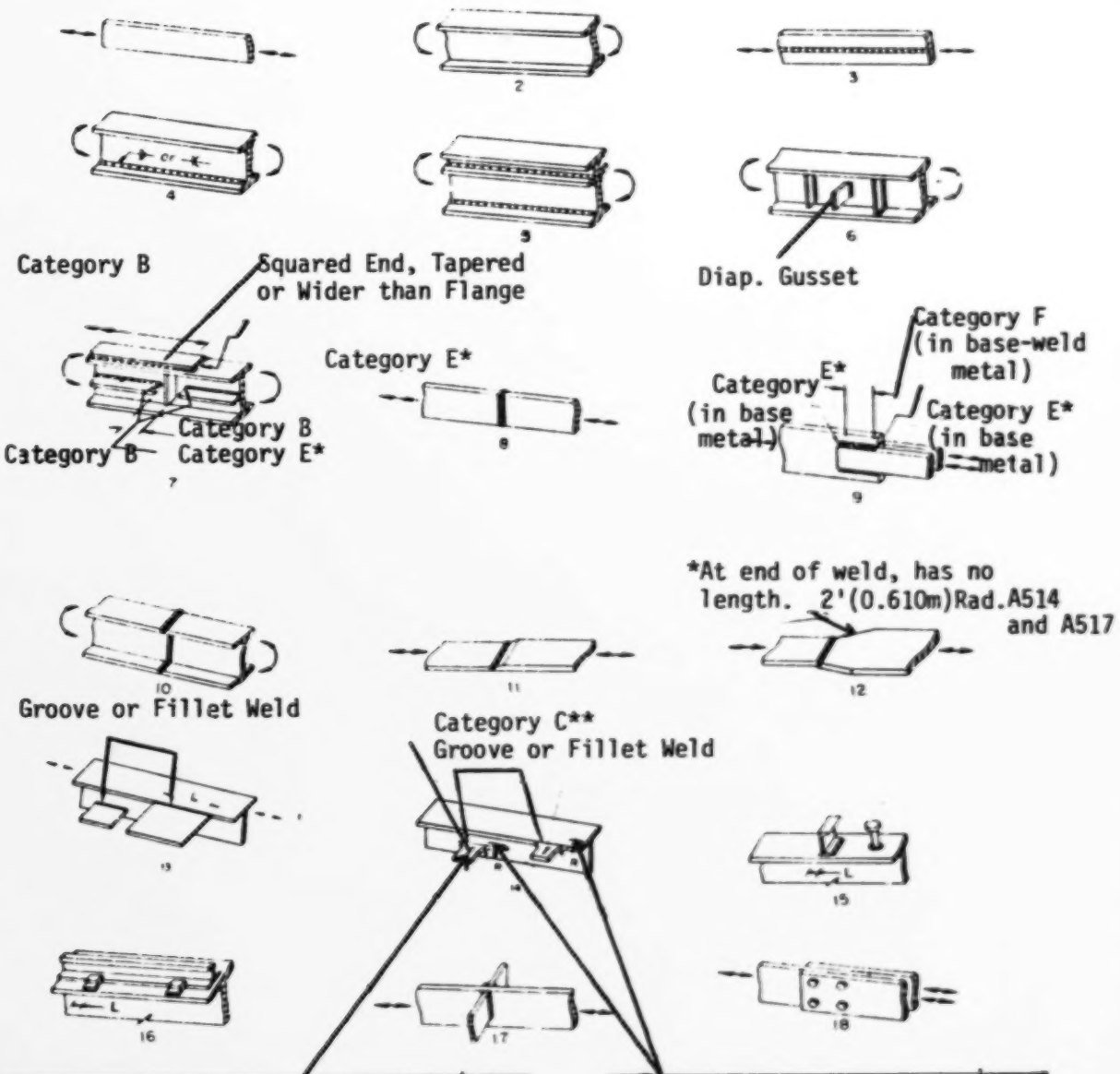
Partial joint penetration groove welds are permitted only between components of built-up main members designed primarily for axial stress, such as in corner and tee joints parallel to the direction of computed stress, in secondary or non-stress carrying members in shoes, etc. Intermittent fillet welds are permitted only to connect stiffeners to beam and girder webs. However, this practice is unacceptable in new construction, and it is strongly recommended that fillet welds on stiffeners be continuous. Fatigue stress range classifications are summarized in Figure 3. The AWS Code specifies all dimensions and tolerances of weld details for complete and partial joint penetration groove welds and for fillet welds as well. Examples of prequalified welds and their sizes are given in Figures 4 and 5 and Table 9.

3.4 Workmanship

3.4.1 General

Workmanship requirements of the AASHTO specifications[12,13] and AWS Code are applicable to all welding prequalified processes and procedures, whether done in the shop or field, and good results can be achieved only by complete cooperation of those who are responsible for all steps in the welding; the designer, welding engineer, welder, fabricator and erector.

Those requirements are directed at solving the main problems that arise when dealing with dynamically loaded structures: avoiding distortion and



Weld Condition *	CAT
Unequal Thickness-Reinf. in place	E
Unequal Thickness-Reinf. Removed	D
Equal Thickness-Reinf. in place	C
Equal Thickness-Reinf. Removed	B

R**	CAT
R ≥ 24"(610mm)	B
24"(610mm) >R > 6"(152mm)	C
6"(152mm) >R > 2"(51mm)	D
2"(51mm) >R	E

* For transverse loading-check transition radius for possible lower category

** Also applies to transverse loading

Figure 3 Illustrative Examples of Fatigue Stress Range Classifications (7)

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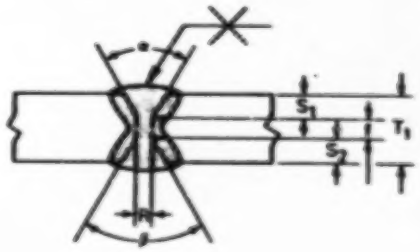
Double V-groove weld (3)
Butt joint (B)



For B-U3c-S only (a)

T_1		S_1
Over	to	
2	2 1/2	1 3/8
2 1/2	3	1 3/4
3	3 5/8	2 1/8
3 5/8	4	2 3/8
4	4 3/4	2 3/4
4 3/4	5 1/2	3 1/4
5 1/2	6 1/4	3 3/4

For $T_1 > 6\ 1/4$, or $T_1 < 2$
$S_1 = 2/3\ (T_1 - 1/4)$

Welding process	Joint designation	Base metal thickness (a) (U = unlimited) T_1	T_2	Groove preparation (a): Root opening, Root face, Groove angle	Tolerances: As detailed	Tolerances: As fit up	Permitted welding positions	Gas shielding for FCAW	Notes
SMAW	B-U3b	U, preferably 5/8 or thicker	–	R = 0 to 1/8 f = 0 to 1/8 α = β = 60°	+1/16, –0 +1/16, –0 +10°, –0	+1/16, –1/8 Not limited +10°, –5°	All	–	C,M
GMAW FCAW	B-U3-GF						All	Not required	A,C,M
SAW	B-U3b-S	1-1/8 min	–	R = 1/8 f = 0 α = β = 60° $S_1 = 2/3\ T_1$, $S_2 = 3/8$ min	+1/16, –0 +1/16, –0 +10°, –0°	+1/16, –1/8 Not limited +10°, –5°	Flat	–	P
SAW	B-L3-S	1-1/2 max	–	R = 0 f = 1/4 max α = 60°, β = 80° $S_1 = 2/3\ (T_1 - 1/4)$, $S_2 = 1/3\ (T_1 - 1/4)$	±0 +0, –1/4 +10°, –0°	+1/16, –0 Not limited +10°, –5°	Flat	–	K
SAW	B-U3c-S	U	–	R = 0 f = 1/4 max α = β = 60° To find S_1 see table above; $S_2 = (T_1 - S_1 + 1/4)$	±0 +0, –1/4 +10°, –0°	+1/16, –0 Not limited +10°, –5°	Flat	–	C

Note A: Not prequalified for gas metal arc welding using short circuiting transfer.

Note C: Gouge root before welding other side.

Note K: Weld root after welding at least one pass on arrow side.

Note M: Double groove welds may have grooves of unequal depth, but the depth of the shallower groove shall be no less than one-fourth of the thickness of the thinner part joined.

Note P: Weld S_2 first with shielded metal arc low-hydrogen electrodes. Root of this weld shall be back gouged. Weld S_1 with single- or multiple-pass submerged arc weld in flat position after manual arc welding is completed on other side.

(a) To obtain equivalent values in cm. Multiply listed values by 2.54.

Figure 4 PREQUALIFIED COMPLETE JOINT PENETRATION GROOVE WELDED JOINTS (11)

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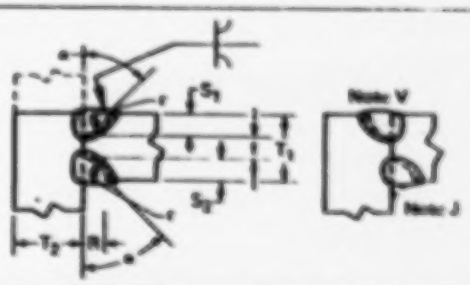
Double J groove weld (S)
Butt joint (B)
T joint (T)
Corner joint (C)



Welding process	Joint designation	Base metal thickness (a) (U = unlimited) T_1	T_2	Groove preparation (a) Root opening, Root face, Groove radius, Groove angle	Tolerances: As detailed	Tolerances: As fit-up	Permitted welding positions	Effective throat (E)	Notes
SMAW	BTC P9	3/8 min. (for bridges 1/2 min)	U	R = 0 to 1/8 f = 1/8 min r = 3/8 α = 45°	+1/16,-0 -0 +1/4,-0 +10°,-0°	±1/16 ±1/16 ±1/16 ±10°,-5°	All	S	E, L, V
GMAW FCAW	BTC P9 GF	3/8 min (for bridges 1/2 min)	U	R = 0 f = 1/8 min r = 3/8 α = 30°	+1/16,-0 -0 +1/4,-0 +10°,-0°	±1/16 ±1/16 ±1/16 ±10°,-5°	All	S	A, E, L, V
SAW	C P9 S	1/2 min (for bridges 5/8 min)	U	α = 20°	+10°,-0°	+10°,-5°	Flat	S	E, V
				R = 0 f = 1/4 min r = 1/2	+0 -0 +1/4,-0	+3/16,-0± ±1/16 ±1/16			
	T P9 S	1/2 min	U	α = 45°	+10°,-0°	+10°,-5°	Flat	S	E, L

Note A: Not prequalified for gas metal arc welding using short circuiting transfer.

Note E: Minimum effective throat (E) as specified on drawings.

Note L: Butt and T joints are not prequalified for bridges.

Note Mp: Double groove welds may have grooves of unequal depth, provided they conform to the limitations of Note E. Also, the effective throat (E), less any reduction, applies individually to each groove.

Note V: For corner joints, the outside groove preparation may be in either or both members, provided the basic groove configuration is not changed and adequate edge distance is maintained to support the welding operations without excessive edge melting.

± Fit-up tolerance, SAW for rolled shapes R may be 5/16 inches in thick plates if backing is provided.

(a) To obtain equivalent values in cm multiply listed values by 2.54.

Figure 5 PREQUALIFIED PARTIAL JOINT PENETRATION GROOVE WELDED JOINTS (11)

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TABLE 9

MINIMUM SIZE OF FILLET WELD FOR BRIDGES

BASE METAL THICKNESS OF THICKER PART JOINED		MINIMUM SIZE OF FILLET WELD			
		AWS Code[11]		AASHTO SSHB [7]	
in	mm	in	mm	in	mm
To 1/2 (inclusive)	12.7	3/16	5	3/16	4.8
Over 1/2 to 3/4	12.7 to 19.1	1/4	6	1/4	6.4
Over 3/4 to 1 1/2	19.1 to 38.2	5/16	8	5/16	7.9
Over 1 1/2 to 2 1/4	38.2 to 57.2	5/16	8	3/8	9.5
Over 2 1/4 to 6	57.2 to 152.4	5/16	8	1/2	12.7
Over 6	152.4	5/16	8	5/8	15.9

providing adequate fracture toughness and fatigue properties of the welded joints.

Increased use of welding and thinner plates used in the fabrication of bridge steel work resulted in bad quality and a greater accelerating increase in costs due to changes of the size and shape of the members. That is why the AWS Code prescribes control of dimensional tolerances, straightness, and flatness of structure shapes and plates.

The importance of avoiding severe notch effects, abrupt discontinuities, or changes in section, and other forms of geometrical constraints is underlined in the AWS Code through special requirements, particularly if the structures are to be subjected to relatively low temperatures.

3.4.2 Preparation of Material

Surfaces and edges to be welded should be smooth, uniform, and free from fins, tears, cracks and other discontinuities which might adversely affect the quality or strength of the weld. Surfaces to be welded and surfaces adjacent to a weld should be free from loose or thick scale, slag, rust, moisture, grease, and other foreign material. But mill scale that can withstand vigorous wire brushing, a thin rust inhibitive coating, or antispatter compound may remain, except for girders where all mill scale should be removed from the surfaces on which flange-to-web welds are to be made.

Strict rules are established for the quality of the edges cut by oxygen cutting and their repair by welding.[12] The roughness of the edges, the types and allowable dimensions of the discontinuities are specified.[12]

Various methods of joint preparation, back gouging, or the removal of unacceptable work or metal are prescribed, including machining, carbon arc cutting, oxygen cutting or gouging, chipping and grinding. The one exception is that oxygen gouging is usually not used on steels that are ordered as quenched and tempered or normalized[12] and for FCMs repair.[13]

3.4.3 Assembly

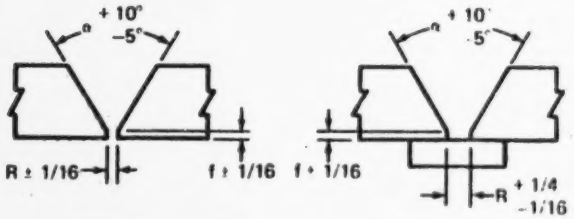
In the assembly of material to be welded, utmost attention should be paid to the correct alignment of the parts. An offset not exceeding 10% of the thickness of the thinner part joined, but not more than 1/8 in. (3.2mm), is permitted where the parts are effectively restrained against bending due to eccentricity in alignment.

Joint configuration and fit-up should be maintained as stipulated in the AWS Code. Special attention should be paid in fitting-up the parts to be joined within tolerances for root opening, root face, and groove angle of the point. Excessive deviation in this respect would constitute grounds for qualification of joints, normally considered prequalified, or rejection of the work.

The root opening is the separation between the members and is used for electrode accessibility to the root of the joint and to maintain a proper root fusion. The root face is to provide an additional thickness of metal at the root of the joint, in order to minimize any burn-through tendency. Dimensions of the root face, root opening, and groove angle are interdependent as well as dependent upon whether a backing or back-gouging is stipulated in the welding procedure specification. The smaller the groove angle, the larger the root opening should be. The AWS Code tolerances in assembly of groove welded butt joints are given in Figure 6. A root opening wider than those permitted but not greater than twice the thickness of the thinner part or 3/4 in (19mm), may be corrected by welding without approval of the engineer.

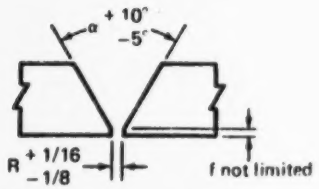
The larger root opening requires the larger root face. Groove welds having root faces usually require back gouging when a 100% penetration weld is to be made. That is why the AWS Code does not specify the tolerances for root

	Root not gouged		Root gouged	
	in.	mm	in.	mm
(1) Root face of joint	± 1/16	1.6	Not limited	
(2) Root opening of joints without steel backing	± 1/16	1.6	+ 1/16 − 1/8	1.6 3.2
Root opening of joints with steel backing	+ 1/4 − 1/16	6.4 1.6	Not applicable	
(3) Groove angle of joint	+ 10 deg − 5 deg		+ 10 deg − 5 deg	



(A) Groove weld without backing – root not gouged

(B) Groove weld with backing – root not gouged



(C) Groove weld without backing – root gouged

NOTES:
1. α groove angle
2. R root opening
3. F root face

Figure 6 WORKMANSHIP TOLERANCES IN ASSEMBLY OF GROOVE WELDED BUTT JOINT.(11)

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faces in this case. The root face is also not recommended when backing is used.

To eliminate fusion defects at the root of the groove welds when using manual shielded metal arc welding without backing, the back side of the joint must be removed down to sound metal by any convenient means: grinding, chipping or carbon-arc gouging, the latter method being the most economical and common in practice. The AWS Code specifies profile dimensions for a groove produced by gouging.

The members to be welded are usually brought into correct alignment by tack welds. Jigs and fixtures are recommended where practical. The AWS Code requirements for the tack welds are the same as for the final welds. However, for single pass tack welds which are remelted and incorporated into continuous submerged arc welds, it is not mandatory to use preheat or to remove discontinuities such as undercut, unfilled craters, and porosity before the final welding starts. The tack welds which are not incorporated into final welds should be removed.

Backing material is commonly used when all welding must be done from one side, or it may be used when the root opening is excessive. Material for the backing should conform to any one of the specifications for base metals. However, the steel backing of welds that are transverse to the direction of computed stress should be removed, since it reduces the fatigue strength of the joint. When the direction of stress is parallel to the backing, or if not subjected to a cyclic stress the backing may be left in place.

3.5 General Requirements of Welding

3.5.1 Weld Profile

The faces of fillet welds which are important in terms of their influence on mechanical, fracture, and fatigue properties of the welded joint may be slightly convex, flat, or slightly concave as specified by AWS Code (Figure

7 a, b, c). Excessive concavity results in a reduction of effective throat of the weld, and excessive convexity can reduce the fatigue life of the joint.

The groove welds (Figure 7 d, e) should be made with slight or minimum reinforcement (not exceeding 1/8 in. (3.2mm) for butt and corner joints) and a gradual transition to the plane of the base metal surface is needed to reduce the risk of fatigue failure.

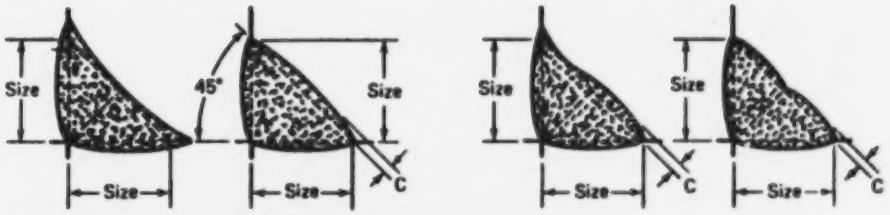
Butt joints between parts subjected to tensile stress are required to be flush or finished to a smooth transition, and the direction of finish grinding should be in the direction of stress if the ANSI roughness value of the ground surface is more than specified by AWS Code.

3.5.2 Other Requirements

In-Process cleaning of the weld is important for performing sound welds in bridge welding. The AWS Code requires removing all slag not only from the successive layers but also from successive beads when multiple layer welding is used. This requirement is also applied to the crater area when welding is resumed after any interruption. After the welding is completed the weld and adjacent area should be brushed clean and painted only after the work has been completed and accepted.

Weld terminations are the most dangerous area in terms of fatigue failure, and should be done by using extension bars and run-off plates which are to be removed upon completion and cooling of the welds. The ends of the weld should be made smooth and flush with the edges of abutting parts.

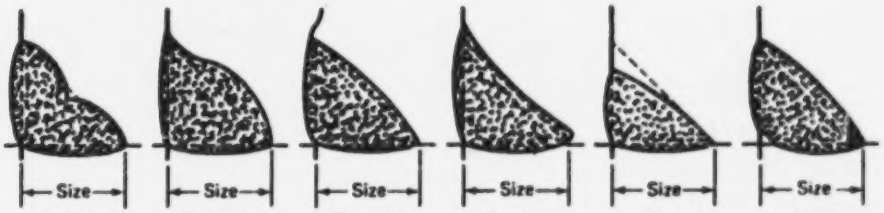
Arc strikes outside the area of permanent welds should be avoided, and cracks or blemishes caused by strikes should be ground flush, otherwise they might become areas of fatigue crack initiation.



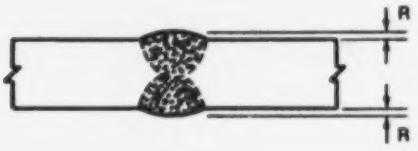
Note: Convexity C shall not exceed 0.1 times actual leg size, or the longer leg in the case of an unequal leg fillet weld, plus 0.06 in. (0.3 mm).

(A) Desirable fillet weld profiles

(B) Acceptable fillet weld profiles

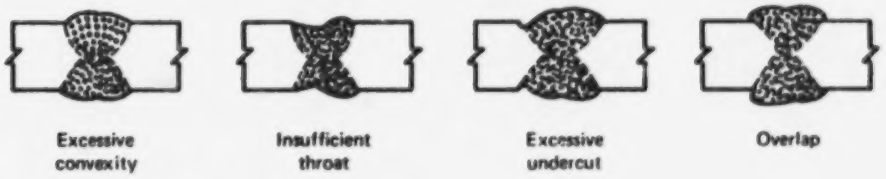


(C) Unacceptable fillet weld profiles



Note: Reinforcement R shall not exceed 1/8 in. (3.2 mm).

(D) Acceptable butt weld profile



(E) unacceptable butt weld profile

Figure 7 ACCEPTABLE AND UNACCEPTABLE WELD PROFILES (11)

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3.5.3 Control of Distortion and Shrinkage

Distortion during fabrication is a serious problem in bridge welding and special measures are taken by the AWS Code to minimize it.

The AWS Code requires that the welding sequence and distortion control program should be developed by the contractor before the start of the welding. All the welds should be deposited in a sequence that will balance the applied heat of welding while the welding progresses. The direction of welding should be from the joint of higher restraint to the joint where they have a greater relative freedom of movement. The joints expected to have larger shrinkage should be welded before the joints with lesser shrinkage. Welding under severe restraint conditions should be carried out continuously to completion in order to keep the temperature of the joint higher than the minimum specified preheat and interpass temperature.

AWS Code specifies permissible variations in straightness, flatness, camfer and so on of welded columns, beams girders, and other bridge members.

3.5.4 Requirements for Repair Welding

The contractor has the option of either repairing an unacceptable weld or removing and replacing the entire weld. The requirements of the repair are the same as for the method originally used. The additional stricter requirements for FCMs repair, in regard to documentation, approval, welding methods, limits of excavation, preheat/interpass temperature, etc. are specified in the Guide Specifications.[13]

The removal of weld metal or portions of the base metal may be done by machining, grinding, chipping, oxygen or air carbon gouging. For quenched and tempered steel, and for FCMs repair, oxygen gouging is not permitted.

Additional metal is deposited by an electrode preferably smaller than that used for making the original weld but not more than 5/32 in. (4.0mm).

The defects which are subjected to repair by welding are the following: excessive concavity of the weld or the crater, undersize welds, undercutting, excessive weld porosity or slag inclusions, incomplete fusion and cracks.

3.6 Welding Technique

3.6.1 Welding Processes

The AWS Code provides the following four welding methods for use as prequalified processes:

Shielded Metal Arc Welding (SMAW)

Submerged Arc Welding (SAW)

Gas Metal Arc Welding (GMAW)

Flux Cored Arc Welding (FCAW)

These welding methods can be used for bridge welding without procedure qualification tests. GMAW and FCAW with multiple electrode, GMAW with short circuiting transfer (GMAW-S) are also permitted for bridge welding, however, they do not have prequalified status and require that the contractor qualify the procedure specifications by tests in accordance with the terms of the AWS Code. As for fabrication, the GMAW, electroslag, and electrogas welding methods are not authorized at all for FCMS.

3.6.2 Filler Metal

In order to perform welds with given mechanical properties, and to make base and weld metals match each other, a proper selection of filler metal is absolutely necessary. Prequalified filler metals for welding complete joint penetration butt welds on bridge members made from various steel, can be found in Standard Specifications for Welding of Structural Steel Highway Bridges[12] and are given in Table 10.

For exposed bare unpainted application of A242 and A588 steel, requiring weld metal with atmospheric corrosion resistance and coloring characteristics similar to that of the base metal, AWS Code specifies the matching filler

TABLE 10

FILLER METAL REQUIREMENTS FOR COMPLETE JOINT PENETRATION BUTT WELDS (12)

Base Metal (1) ASTM Designation	WELDING PROCESS (2) Shielded Metal Arc	Submerged Arc	Gas Metal Arc	Flux Cored Arc
A36 (3)	AWS A5.1 E 7016.18 or 28	AWS A5.17 (4) F 62, F 63 or F 64-EXXXX	AWS A5.18 E70S- 1B 2,3,6 or E70U-1	AWS A5.20 (5) E60T-8
A242, A441 A572 Gr,42,45 & 50 A588	AWS A5.1 E7016.18 or 28	AWS A5.17 (4) F72 F73 or F74 EXXXX	AWS A5.18 E70S-18 2,3,6 or E70U-1	AWS A5.20 (5) E70T-1 5 or 6
A514/A517	AWS A5.5 E11018-M	Grade F110	Grade E110S	Grade E110T

Use of same type filler metal having next higher mechanical properties as listed in the AWS Specifications is permitted.

(1) In joints involving base metals of different yield points or strengths, filler metal electrodes applicable to the lower strength base metal may be used.

(2) When welds are to be heat stress relieved the deposited weld metal shall not exceed 0.05 percent vanadium.

(3) AWS A5.1-E6010 or E6011 electrodes may be used for welding A36 steel not more than 1 inch (25.4mm) thick.

(4) An E61-EXXXX or F71-EXXXX classification may be used if a representative sample of the material delivered for a project is tested for conformance with Table 4 of AWS A5.17.

(5) Electrodes of other classifications in the specification will be acceptable for use if the manufacturer's certificate of test shows that the mechanical properties (tensile strength, yield strength, and elongation), and the impact properties meet the requirements of the classification or classifications, listed.

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materials which are given in Table 11. In multiple-pass welding at least two layers on all exposed surfaces and edges are to be deposited by the filler metal from Table 11. Special requirements are specified[13] in terms of hydrogen contents in consumables for flux-cored and submerged arc welding methods of FCMs.

3.6.3 Preheat and Interpass Temperature Requirements

For prequalified welding methods, base and filler material minimum preheat and interpass temperatures are specified in Standard Specification for Welding of Structural Steel Highway Bridges[12] and are shown in Table 12. Keeping the preheat and interpass temperature in accordance with that specified in Table 12 enables the welds to be free from crack formation. Temperatures above the minimum shown may be required for highly restrained welds and for FCMs.[13]

3.6.4 Stress Relief Heat Treatment

Where required by the contract drawing or specifications welded assemblies should be stress-relieved by heat treating. This is done in order to reduce residual stresses and to make their level uniform, to retain dimensional stability during machining and to prevent stress corrosion. However, stress relieving of weldments of A514 and A517 is not generally recommended because it may actually impair weld metal and heat-affected zone toughness, and intergranular cracking may sometimes occur in the grain-coarsened region of the HAZ. The requirements for stress relief treatment are specified in AWS Code.

3.6.5 Procedures for Prequalified Welding Methods

The AWS Code provides general requirements for the welding procedure of four prequalified welding methods: SMAW, SAW, GMAW, and FCAW. The Code specifies the requirements for storage and atmospheric exposure times for covered low hydrogen electrodes, size of electrode and wire, welding position, and condition for each welding method, particulars of each welding method for single pass or multiple-pass welds.

Table 11 (12)
Filler metal requirements for exposed bare applications of ASTM A242 and A588 steel

Welding process			
Shielded metal arc	Submerged arc	Gas metal arc	Flux cored arc[2,4]
AWS A5.5	AWS A5.23	AWS A5.28	
E8016 or 18-G[1,2]	F7X-EXXX-W[2,3]	ER80S-G[2,4]	
E8016 or 18-B1[2]	F7X-EXXX-B1[2,3]	ER80S-B2[2,3]	62 ksi min YP
E8016 or 18-B2[2]	F7X-EXXX-B2[2,3]	ER80S-B2L[2,3]	(430 MPa)
E8015 or 18-B2L[2]			72 ksi min TS
E8016 or 18-C1	F7X-EXXX-Ni1[3]	ER80S-Ni1[3]	(495 MPa)
E8016 or 18-C2	F7X-EXXX-Ni2[3]	ER80S-Ni2[3]	Elon. 18% min
E8016 or 18-C3	F7X-EXXX-Ni3[3]	ER80S-Ni3[3]	

1. Deposited weld metal shall have the following chemical composition: C, max %, 0.12; Mn, %, 0.50/1.30; P, max %, 0.03; S, max %, 0.04; Si, %, 0.35/0.80; Cu, %, 0.30/0.75; Ni, %, 0.40/0.80; Cr, %, 0.45/0.70.
2. Deposited weld metal shall have a minimum impact strength of Charpy V-notch 20 ft • lb (27.1 J) at 0° F (−18° C) (only applied to bridges).
3. The use of the same type of filler metal having next higher mechanical properties as listed in AWS specifications is permitted.
4. Deposited weld metal shall have a chemical composition the same as that for any one of the weld metals in this table for the shielded metal arc welding process.

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Table 12

Minimum Preheat And Interpass Temperature (12) Degrees F (C) (1),(3)

Welding Process (Base Metal)	Thickness of Thickest Part at Point of Welding Inches (mm)			
	To 3/4 in. Incl. (19.1)	Over 3/4 in. to 1½ in. Incl. (19.1 to 38.2)	Over 1½ in. to 2½ in. Incl. (38.2 to 63.5)	Over 2½ in. (63.5)
Shielded Metal Arc Welding with Low Hydrogen Electrodes; Submerged Arc Welding; Gas Metal Arc Welding; or Flux Cored Arc Welding (ASTM A36 (2); A242; A441; A572 Grades 42, 45 and 50; A588)	50 (10)	70 (21.1)	150 (65.6)	225 (107.2)
Shielded Metal Arc Welding with Low Hydrogen Electrodes; Submerged Arc Welding with Carbon or Alloy Steel Wire, Neutral Flux; Gas Metal Arc Welding; or Flux Cored Arc Welding (ASTM A514/A517)	50 (10)	125 (51.7)	175 (79.4)	225 (107.2)
Submerged Arc Welding with Carbon Steel Wire, Alloy Flux (ASTM A514/A517)	50 (10)	200 (93.3)	300 (148.9)	400 (204.4)

1. Welding shall not be done when the ambient temperature is lower than zero F (-17.8C). When the base metal is below the temperature listed for the welding process being used and the thickness of material being welded, it shall be preheated (except as other wise provided) in such manner that the surfaces of the parts on which weld metal is being deposited are at or above the specified minimum temperature for a distance equal to the thickness of the part being welded, but not less than 3 inches (76.2mm) both laterally and in advance of the welding. Preheat and interpass temperatures must be sufficient to prevent crack formation. Temperature above the minimum shown may be required for highly restrained welds. For ASTM A514/A517 steel the maximum preheat and interpass temperature shall not exceed 400°F (204.4C) for thicknesses up to 1½ inches(38.2mm) inclusive, and 450°F (232.2°C) for greater thicknesses. Heat input when welding ASTM A514/A517 steel shall not exceed the steel producer's recommendation. Welding shall be carried continuously to completion or to a point that will insure freedom from cracking before the joint is allowed to cool below the minimum specified preheat and interpass temperature.

2. For A36 steel to one-inch thickness, inclusive, when welded with other than low-hydrogen electrodes, the minimum preheat and interpass temperature shall be 150°F (65.6°C).

3. In joints involving combinations of base metals, preheat shall be as specified for the higher strength steel being welded.

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3.7 Qualification

As it was mentioned above the prequalified joint welding procedures are exempt from tests or qualifications.* AWS Code establishes the list of requirements which should be met by those procedures to be considered prequalified. All prequalified joint welding procedures to be used should be prepared by manufacturer, fabricator, or contractor as written procedure specifications to be available for authorized examination. Except for prequalified ones, all other joint welding procedures as well as all the welders , which are to be employed under AWS Code, should be qualified by the tests prescribed by the AWS Code.

3.7.1 Joint Procedure Qualification

It is the manufacturer's or contractor's responsibility to conduct the tests prescribed by the AWS Code. Sometimes changes should be made in prequalified or contractor specification and the AWS Code provides special rules to establish which changes are to be considered essential and will require a new procedure qualification. These changes may refer to the base material within those covered by prequalification status, filler material, joint type and geometry, welding position, welding parameters and by others for each of the prequalified welding methods.

For example, a new qualification is required if there is an increase in filler metal strength level (but not vice versa) for any of the welding methods; an increase of electrode diameter by more than 1/32 in. (1mm) over that used in the procedure qualification; a change of more than 10%, 7% and 15% above or below the specified amperage, voltage and travel speed,

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* except all groove and fillet weld procedures for weld metal with a minimum specified yield point of over 90 ksi, (620.527 MPa)[12] and for FCMs, for which extra rules are established by the Guide Specifications. (13)

respectively for SAW; a change in method of shielding not covered by AWS A5.18 or A5.28 specifications for GMAW or FCAW; a decrease of more than 25°F (13.9°C) in the minimum specified preheat or interpass temperature and so on.

In order to determine the mechanical properties and soundness of welded joints made under a given procedure specification AWS Code specifies types of tests. For example, for groove welds the reduced section tension test and root, face, and side bend tests are specified. All-weld-metal and impact tests may be required for electrogas and electroslag welding methods. For partial joint penetration groove welds the macroetch test is required. In addition radiographic or ultrasonic testing is specified for groove welds. For fillet welds only 3 tests are specified: macroetch, side-bend and all-weld-metal tests.

The AWS Code gives all details concerning the number, type, and preparation of the samples, as well as the methods of their testing.

3.7.2 Welder Qualification

In order to determine a welder's ability to produce sound welds, the AWS Code establishes special tests. If qualification is established for one of the steels permitted by the Code, it is considered as qualification to weld and tack weld any of the other steels.* A welder is qualified not only for material to be welded but for a particular welding process and welding position.

For example, a change in position of welding or welding process requires requalification of the welder. Even the omission of backing material in complete joint penetration welds made from one side will entail requalification.

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* except ASTM A514 & A517 steels for which the evidence must be presented, satisfactory to the Engineer, that each welder, operator and tacker has had at least three months satisfactory experience welding each steel.(12)

3.8 Nondestructive Inspection of Bridge Welds

3.8.1 General

Inspection of bridge welds during fabrication is done to ensure that the previously discussed AWS Code requirements are met for:

- o Welder/Tacker qualification,
- o Welding procedures preparation and equipment, and
- o Materials.

This inspection as governed by the AWS Code falls into two categories. They are:

1) Fabrication/erection inspecting and testing, and
2) Verification inspection and testing.

The first category is the responsibility of the fabricator while the second is the prerogative of the owner.

In addition to ensuring that properly qualified welders are using code acceptable procedures on properly prepared materials, various nondestructive tests are applied, primarily to completed weldments to ensure that the welds are free of flaws within the limits set by the codes. The ultimate objective of these procedures is to ensure that the fabricated structure fulfills its intended purpose in a safe and reliable manner within reasonable economic limits.

All of the nondestructive testing techniques have limitations and these limitations are in general, taken into consideration by the codes both in terms of applicability of a method and acceptance limits for the various flaw types.

Inspection and repair of weldments is a major cost item for welded fabrication. In general, the application of more stringent acceptability requirements are not looked upon favorably by fabricators. The repair of

weldments can be very expensive and if careful procedures are not observed, such repair can be damaging to the weld. This damage can take the form of increased residual stress as well as microstructural damage.

A major effort of this program is to provide improved inspection for welded bridge members. This effort makes use of a new NDT method that as yet has no official AWS/ASME code acceptance. However, a new ASTM Standard Practice (E749-80) has been accepted by the membership and is scheduled for official publication in the fall of 1980. The new method, in-process acoustic emission monitoring has shown great potential for improved sensitivity to the most serious weld flaws (cracks). Additionally, the method has considerable economic attractiveness, allowing flaw detection and repair to be accomplished during the welding process. AE also is generally easier to apply and requires less operator skill than most traditional NDT methods because of its adaptability to computerized signal processing techniques.

In addition to reducing repair costs, with the use of AE, damage to the weld is minimized because the weld can be repaired during fabrication using high reliability welding processes eliminating the need to use post weld repair techniques on cold plates that lead to improper weld repair.

Thus, AE should provide the ultimate payoff: improved weld integrity coupled with lower production cost.

The following subsections summarize the code requirements for the various NDT methods in current usage for bridge fabrication welding. These methods will be applied to the test welds produced during this program to provide correlation for the acoustic emission data.

Each subsection discusses the applicability of the method along with the method's strengths and weaknesses.

3.8.2 Applicable Codes

All codes for the inspection and quality control of highway bridge welding originate in AASHTO "Standard Specification for Highway Bridges".[14] These specifications primarily refer to the AWS (D1.1-80) Structural Steel Welding Code[15] as the main reference for weld inspection standards. However, this code is modified in part by AASHTO "Standard Specifications for Welding of Structural Steel Highway Bridges".[16] The modified AWS Code specifies inspection accept-reject criteria and procedures to be used for visual, magnetic particle, dye penetrant, radiographic and ultrasonic nondestructive inspection techniques. The AWS Code refers to but does not contain all prescribed inspection procedures. Procedures for magnetic particle, dye pentrant and radiographic inspection are specified by the AWS Code to follow American Society for Testing and Materials (ASTM) standard recommended practices.[17] The accept-reject criteria used are prescribed by the AWS Code or depend upon individual customer/vendor requirements. The following Table 13 references the applicable codes as they relate to the various nondestructive inspection methods.

3.8.3 Personnel Qualifications

The qualification procedures for inspection personnel are also specified in the AWS Code. The Code states that personnel are to be qualified in accordance with the American Society for Nondestructive Testing Recommended Practice No. SNT TC-1A.[18] The Recommended Practice contains the general framework for a qualification and certification program. The document also provides recommended educational experience and training requirements for the different nondestructive testing methods. Supplementary documents include the materials necessary for conducting examinations necessary for personnel certification at each level (I, II, III). The AWS Code further specifies

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Table 13 RELATION OF NONDESTRUCTIVE INSPECTION TECHNIQUES TO APPLICABLE CODES

INSPECTION METHOD \ APPLICABLE CODE	AASHTO/SSHB	AASHTO/SS WS SHB	AWS	OTHER
VISUAL	X	X	X	
MAGNETIC PARTICLE	X	X	X	ASTM E 109
DYE PENETRANT	X		X	ASTM E 165
RADIOGRAPHY	X	X	X	ASTM E 94 ASTM E 142
ULTRASONIC	X	X	X	

that only individuals qualified for NDT level I and supervised by personnel certified NDT level II or III,or personnel qualified for NDT level II or III may perform nondestructive inspections.

3.8.4 Nondestructive Inspection Techniques

The methods of nondestructive testing applied to welded highway bridges are visual, radiographic,ultrasonic, magnetic particle, and dye penetrant. Knowledge of the principles and procedures of these methods is essential to ensure that the operators are satisfactorily performing and that test results are correctly interpreted. The information in the following subsections describes rudiments of these methods as applied to welded bridge inspection.

3.8.4.1 Visual Inspection

Visual inspection is the primary quality control tool used by the contractor during fabrication. This fabrication/erection inspection must assure compliance with code standards for material, quality, workmanship, etc. An additional check is provided by the owner's verification inspections.

A detailed description of the code requirements for these items has already been discussed in previous sections. In addition to assuring compliance in these areas during fabrication, visual inspection when applied to completed welds is a valuable NDT method for detection of welding flaws. The following list summarizes those aspects of quality control of bridge welding that can be accomplished by visual inspection.[19]

1. Base metal

   Determine the soundness of base metal and the absence of gross areas of laminations and inclusions on prepared weld joint edges,as well as the correctness of material dimensions.

2. Filler metal

   a. Determine that the electrodes used conform to the requirements of the welding procedure specification, that they have not been damaged by rough handling or improper storage, and that they are properly dried.

   b. Determine that the proper electrodes are employed for each welding position, kind of current and other conditions.

3. Preparation of base metal and fitting for welding

   a. Determine that the edge preparation of base metal at welded joints is done with sufficient accuracy,and that the metal has not been kinked or buckled in handling in such a way as to prevent proper fitting.

   b. Determine that the fit-up jointsare done within proper tolerances.

   c. Determine that the geometry of each joint conforms to the approved welding procedure and is located as called for on the drawings.

4. Execution of the welding

   a. Determine that the approved welding procedures are being followed without deviation by checking welding current, position of welding, use of preheat, etc.

   b. Periodically observe the work of each welder and welding operator and spot-check his ability.

5. Examination of finshed welding

   a. Examine every welded joint for compliance with the working drawings with respect to size, length, and location of welds, and see that no welds have been omitted or added.

b. Examine the welding for compliance with standards of workmanship with respect to the contour of the weld surface, surface defects, craters, undercutting, overlapped edges of welds, cracks, etc.

c. Require corrective measures to be taken wherever necessary, in compliance with the methods prescribed by the specifications.

Visual inspection may be used both as an in-process quality control tool or as an adjunct to other NDT methods for final weld inspection.

Acceptance levels for visual inspection of finished welds generally follow those of dye penetrant inspection. These levels do not permit any cracks or lack of fusion,but a specified minimum amount of porosity (AWS 9.25.1) In addition, weld bead appearance must follow general specifications found in the AWS Code. Examples of acceptable weld profiles are found in AWS 3.6.

The major limitation to this method of inspection lies in its inherent subjective nature. The quality and consistancy that is required for visual inspection can only be gained through experience. Therefore,an experienced inspector is required that is very familiar with the welding methods and materials as they relate to highway bridge welding. Additionally, sub-surface flaws are not detected.

3.8.4.2 Magnetic Particle Inspection

Magnetic particle inspection methods are used for the detection of cracks and other discontinuities at or near the surface of ferromagnetic materials. In this method of inspection, a magnetic field is induced in the weld under test. If there is a defect in the weld section, local magnetic poles will be formed creating a leakage of flux across the discontinuity. If finely divided magnetic particles are applied to the surface they will be attracted by the local magnetic poles and build up across the defect. When

surplus particles are removed, the remaining particles will show the pattern of the defect. The maximum pattern is obtained when the path of the flux is normal to the lengthwise dimension of the defect. Subsurface defects, if not too deep, will also cause leakage of flux, but the magnetic field will be broader and weaker. Advantages of magnetic particle inspection methods are as follows:

- o Rapid, simple, and reliable for finding surface or near surface cracks.
- o No limitation on the size or shape of the part being tested.
- o Relatively low operator skill required.
- o Capable of detecting discontinuities which are filled with non-permeable foreign materials.
- o Relatively inexpensive.

The magnetic particle technique is applicable to bridge weldments primarily as a in-process inspection technique as are liquid penetrant and visual inspection. It can also be used as a final inspection technique as permitted by AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges. This Code primarily excludes its use for final inspection of groove welds in main members. However, AWS Section 6.7.9 (Modified by AASHTO) requires that all fillet welds and longitudinal butt weld splices on beam or girder webs shall be tested by magnetic particle methods. Detailed recommended practices for bridge weld inspection are stated by Section 6.7.5, AWS Structural Welding Code, to follow ASTM E 109 "Standard Method for Dry Powder Magnetic Particle Inspection". These standard methods provide information as to apparatus, surface preparation, inspection medium and magnetization techniques (yoke and prod type).

Magnetic particle indications are generally grouped into 3 classes: surface defects, subsurface defects, and nonrelevant magnetic conditions.

Surface defects produce strong distinct patterns and include cracks, slag inclusions and undercuts. Subsurface defects produce broad fuzzy indications that include subsurface cracks, slag inclusions, incomplete fusion and inadequate joint preparation. Nonrelevant indications can be caused by changes in permeability in the heat affected zone of the weld and irregularities on the surface of the weld. Acceptance standards for defects relating to bridge weldments are stated in Section 9.25.2 AWS Structural Welding Code (AWS D1.1-80). These standards prohibit any cracks or incomplete fusion. Porosity or fusion-type discontinuities are limited to the specifications shown in Figure 8 for tension welds and Figure 9 for compression welds. This inspection standard has similar requirements to radiographic inspection, and incorporates flaw evaluation criteria for visual and dye penetrant inspection.

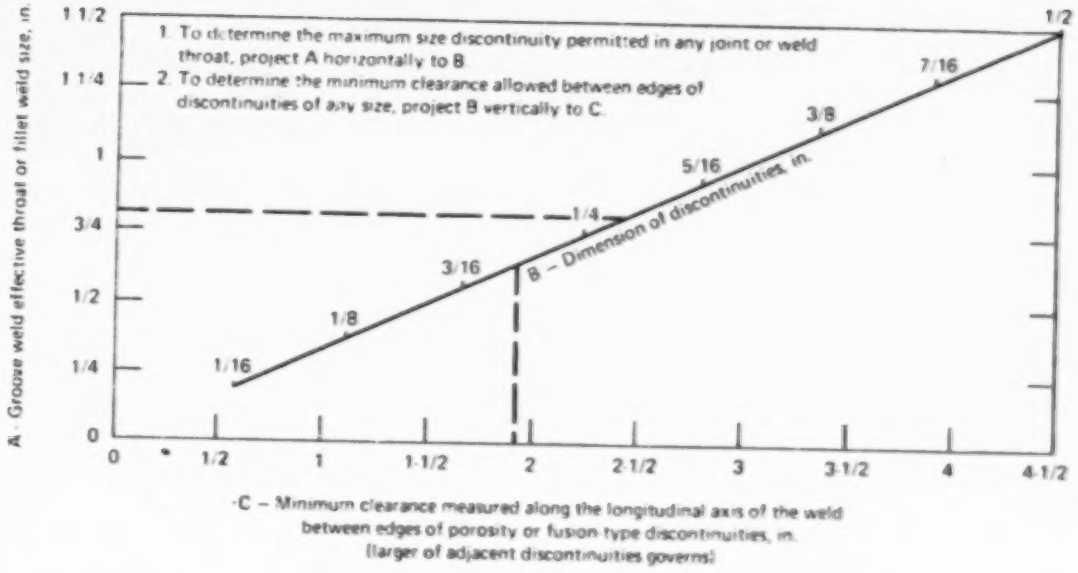
Magnetic particle inspection is subject to the following limitations:

- o Only partially effective in locating subsurface discontinuities and is dependant on defect orientation.
- o Excessively high currents may be required in inspecting large or heavy sections.
- o Demagnetization following inspection usually is necessary.

Magnetic particle inspection methods, like dye penetrant inspection methods, due to their limitations in detecting only surface and near-surface defects, cannot be used as a final inspection technique in many cases, as specified by the code (AASHTO 6.7.1 and 6.7.8).

3.8.4.3 Liquid Penetrant Inspection

Liquid penetrant inspection methods provide a means for the detection of discontinuities that are open to the surface. Liquid penetrant inspection is a relatively simple process. The weld to be inspected is first cleaned



Note: Adjacent discontinuties, spaced less than the minimum spacing required, shall be measured as one length equal to the sum of the total length of the discontinuities plus the length of the space between them and evaluated as a single discontinuity To obtain equivalent values in cm multiply above values by 2.54.

Figure 8 **Weld quality requirements for discontinuities occurring in tension welds (limitation of porosity and fusion-type discontinuities)** (20)

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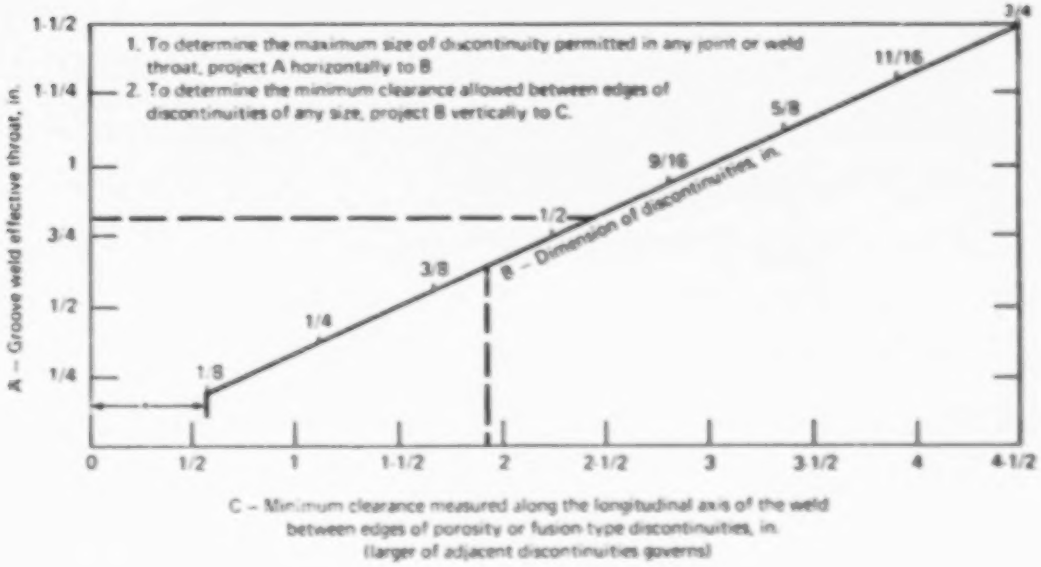

* The maximum size of a discontinuity located within this distance from an edge of plate shall be 1/8 in. (3.2mm), but a 1/8 discontinuity must be 1/4 in. (6.4mm) or more away from the edge. The sum of discontinuities less than 1/8 in. in size and located within this distance from the edge shall not exceed 3/16 in. (4.8mm). Discontinuities 1/16 in. to less than 1/8 in. will not be restricted in other locations unless they are separated by less than 2L (L being the length of the larger discontinuity); in which case, the discontinuities shall be measured as one length equal to the total length of the discontinuities and space. To obtain equivalent values in cm multiply above values by 2.54.

Figure 9 **Weld quality requirements for discontinuities occurring in compression welds (limitations of porosity or fusion-type discontinuities)** (21)

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thoroughly. A liquid penetrant is applied evenly over the surface of the weld being tested and allowed to enter any defects which are open to the surface. After a suitable dwell time, the excess penetrant is removed and the weld is dried. The developer, a light colored, powdered absorbant material, is then applied to the surface. This material acts as a blotter and draws out a portion of the entrapped penetrant out of the discontinuity, staining the developer. The test weld is then inspected visually to determine the presence or absence of indications. After the inspection is completed, the weld must be cleaned thoroughly to remove penetrant and developer. The advantages of this inspection method are:

- o Minimum equipment requirements,
- o Applicable to parts of all sizes and geometries,
- o Fast and economical,
- o Requires no specialized training, once techniques have been mastered.

APPLICABILITY

Liquid penetrant techniques are applicable to bridge welding for the detection of discontinuities that are open to the surface. The method is applicable to in-process and final inspection. Detailed recommended practices for bridge weldment inspection are stated by Section 6.7.6 AWS Structural Welding Code (AWS D1.1-80) to follow ASTM Standard Recommended Practice for Liquid Penetrant Inspection Method (ASTM E165).

ACCEPTANCE LEVELS

Interpretation of the results of the method require relatively little experience or training. Surface flaws relating to cracks, lack of fusion and porosity are readily detected in welds. Crack and lack of fusion usually show up as a continuous line of penetrant indication. The width and brightness

will depend on the volume of the crack. Porosity at the weld surface will show up as small dots. Acceptance standards for defects on bridge weldments are stated in Section 9.25.4, AWS Structural Welding Code (AWS D1.1-80). Briefly, these standards do not allow any surface cracks or incomplete fusion and state the sum of the diameters of piping porosity can not exceed 3/8-inch (.95cm) in any linear inch of weld or 3/4-inch (1.9cm) in any 12-inch (30.5cm) length of weld. These acceptance standards are similar to those for magnetic particle visual inspection.

LIMITATIONS

Major disadvantages of this method are:

- o Limited to the detection of surface defects.
- o Defect depth cannot be determined.
- o Parts must be cleaned well before and after inspection.

The limitation of this method to detection for surface defects requires it to be used only in conjunction with an inspection method that would identify subsurface defects. Acceptable inspection methods for subsurface flaw detection would include radiography or ultrasonics.

3.8.4.4 Radiographic Inspection

Radiography involves the use of penetrating radiation to inspect the internal structure of a weld. Radiation energy is attenuated as a function of material thickness, density, and chemical composition. Since many defect conditions represent an apparent change in one or more of these factors, they cause a predictable change in transmitted or reflected radiation. For example, porosity in a weld represents an apparent decrease in material thickness, and permits the transmission of more radiation than a homogeneous weld. X-ray or gama-ray sources are commonly used in radiography, as well as film, to record the image of the test result. Other radiation sources (alpha, beta and neutron)

and imaging techniques are available but do not represent standard techniques. Radiography has many advantages that are not seen in other NDT methods, these include:

- o Permanent records are obtained showing defects in relation to significant weld features.
- o The image nature of the record greatly aids interpretation of results and allows flaws to be discerned while weld features can be, to some extent, ignored.

APPLICABILITY

Radiography represents a standard inspection technique for bridge weldment inspection. In many cases, for example, groove welds in main members, radiography is required by code (AWS Structural Steel Welding Code 6.7.1 as modified by AASHTO Standard Specifications for Welding Structural Steel Highway Bridges). Basic radiographic methodology is specified by AWS 6.8.1 to conform to ASTM E94 "Standard Recommended Practice for Radiographic Testing". Modifications to these procedures can be found in AWS Structural Steel Welding Code 6.10. These modifications particularly address to penetrameter design, placement,and resultant image quality.

ACCEPTANCE LEVELS

Interpretation is the most difficult step in radiographic procedure. The interpreter must be familar with highway bridge welding processes and materials, as well as what constitutes a rejectable defect. He must also be well trained in the interpretation of radiographic results. Basic procedures for radiographic interpretation,as well as reference radiographs,can be found in ASTM E390 "Standard Reference Radiographs for Steel Fusion Welds". Defect criteria specifically relating to bridge weldingare specified in

AWS 9.25.2, Structural Welding Code. The code prohibits any cracks, and limits fusion type discontinuities as specified in Figure 8 for tension welds and Figure 9 for compression welds. Defects shall be repaired in accordance with AWS 3.7. Furthermore, defect criteria relating to the radiographic procedure can be found in AASHTO Standard Specifications for Welding Structural Steel Highway Bridges, as modifications to AWS Structural Steel Welding Code 6.7.1.

LIMITATIONS

The major disadvantages of radiography include:

- o Access to both sides of a part is required.
- o Sensitivity to planar defects is a strong function of defect orientation.
- o High initial investment and operating costs.
- o Need for highly trained interpreter.

3.8.4.5 Ultrasonic Inspection

Ultrasonic testing involves the observation of the interaction of high frequency sound waves with the weld being inspected. The ultrasonic wave travels with little loss in homogeneous materials, but is interrupted and reflected by weld discontinuities. The ultrasonic beam is usually generated and received in the material through piezoelectric transducers. A couplant, such as glycerine or oil, must be used to transfer mechanical energy from the transducer to the surface of the material. Reflected signals are amplified and displayed on a suitable cathode ray tube. From this display, flaw size and position can be determined.

Major advantages of ultrasonic inspection include:

- o High sensitivity; very small defects can be detected under favorable conditions.
- o Penetrating power; thicknesses up to 30 inches (762 mm) in steel can be inspected.
- o Accurate defect location.
- o Fast response.
- o Minimum accessibility requirements; access to only one surface is required.

APPLICABILITY

Ultrasonic inspection is the most commonly accepted complimentary volumetric inspection technique to radiography. Under AWS codes (modified by AASHTO) ultrasonic inspection is an acceptable alternate method to radiography in most cases. Situations could easily arise where ultrasonic testing is the only method of inspection available. For example, if access to only one side of a weld exists, radiography would be impossible. Procedures for ultrasonic inspection are specified in AWS Structural Steel Welding Code (6.13 thru 6.25) with some modifications by AASHTO 6.7.8 Standard Specifications for Welding of Structural Steel Highway Bridges. These procedures cover equipment, reference standards, and calibration, as well as testing and flaw evaluation procedures. Although not code specified, additional information pertaining to procedure can be found in ASTM E164 "Standard Recommended Practice for Ultrasonic Contact Examination of Weldments".

ACCEPTANCE LEVELS

Interpretation of results from ultrasonic inspections can be subjective. A skilled operator with experience in weld inspection is required. Procedures for flaw size evaluation are specified by AWS Structural Welding Code (6.23)

for straight beam as well as angle beam testing. The AWS code also contains sample defect evaluation forms for this purpose (AWS Appendix E Part C). Basic acceptance-rejection criteria for detailed flaws are also code specified (AWS 9.25.3) and are illustrated in Table 14 for welds under tension and Table 15 for welds under compression. Furthermore, defect criteria relating to modifications of ultrasonic inspection procedures are specified in AASHTO Standard Specification for Welding of Highway Bridge Structures as modifications to AWS 6.7.8.

LIMITATIONS

Ultrasonic inspection is at best subjective, requiring a highly skilled operator. It is also prone to over-inspection because of the nature of a weld joint to reflect sound for many different reasons. High deposition rate processes tend to create large grains with heavy grain boundaries. The large grains cause high attenuation, requiring higher receiver gain, causing more background noise. Some grain boundaries can be almost as good a reflector as tight cracks, which again cause added noise to the system. Both of these factors reduce the system sensitivity to cracks and crack-like defects. Furthermore, orientation and shape of a defect may cause confusing indications and result in a loss of a defect signal. The defect signal varies,as the angle of reflectance from the defect changes and may cause the reflection to be directed away from the transducer. Similarly, a defect having a rough surface tends to scatter the reflected energy, while a smooth surface defect reflects a more direct ultrasonic wave.

Table 14

Ultrasonic Acceptance Rejection Criteria (22)

Tension Welds

Flaw severity class	Weld thickness and search unit angle										
	5/16 to 3/4	>3/4 to 1-1/2	>1-1/2 to 2-1/2			>2-1/2 to 4			>4 to 8		
	70°	70°	70°	60°	45°	70°	60°	45°	70°	60°	45°
Class A	+10 & lower	+8 & lower	+4 & lower	+7 & lower	+9 & lower	+1 & lower	+4 & lower	+6 & lower	−2 & lower	+1 & lower	+3 & lower
Class B	+11	+9	+5 +6	+8 +9	+10 +11	+2 +3	+5 +6	+7 +8	−1 0	+2 +3	+4 +5
Class C	+12	+10	+7 +8	+10 +11	+12 +13	+4 +5	+7 +8	+9 +10	+1 +2	+4 +5	+6 +7
Class D	+13 & up-	+11 & up	+9 & up	+12 & up	+14 & up	+6 & up	+9 & up	+11 & up	+3 & up	+6 & up	+8 & up

Class A (large flaws)
Any indication in this category shall be rejected (regardless of length).

Class B (medium flaws)
Any indication in this category having a length greater than 3/4 inch (19 mm) shall be rejected.

Class C (small flaws)
Any indication in this category having a length greater than 2 in. (51 mm) in the middle half or 3/4 inch (19 mm) length in the top or bottom quarter of weld thickness shall be rejected.

Class D (minor flaws)
Any indication in this category shall be accepted regardless of length or location in the weld.

Scanning levels	
Sound path	Above zero reference, dB
to 2-1/2 (64 mm)	20
> 2-1/2 to 5 (64-127 mm)	25
> 5-10 (127-254 mm)	35
> 10 to 15 (254-381 mm)	45

Notes:

1. Class B and C flaws shall be separated by at least 2L, L being the length of the longer flaw, except that when two or more such flaws are not separated by at least 2L, but the combined length of flaws and their separation distance is equal to or less than the maximum allowable length under the provisions of Class B or C, the flaw shall be considered a single acceptable flaw.
2. Class B and C flaws shall not begin at a distance less than 2L, from the end of the weld, L being the flaw length.
3. Flaws detected at "scanning level" in the root face area of complete penetration double groove weld joints shall be evaluated using an indication rating 4 dB more sensitive than that described in 6.19.6.5 when such welds are designated as "tension welds" on the drawing (subtract 4 dB from the indication rating "d").
4. To obtain equivalent values in cm multiply listed values by 2.54.

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Table 15

Ultrasonic Acceptance Rejection Criteria[23]
Compression Welds

Flaw severity class	Weld thickness and search unit angle										
	5/16 to 3/4	>3/4 to 1-1/2	>1-1/2 to 2-1/2			>2-1/2 to 4			>4 to 8		
	70°	70°	70°	60°	45°	70°	60°	45°	70°	60°	45°
Class A	+5 & lower	+2 & lower	−2 & lower	+1 & lower	+3 & lower	−5 & lower	−2 & lower	0 & lower	−7 & lower	−4 & lower	−1 & lower
Class B	+6	+3	−1 0	+2 +3	+4 +5	−4 −3	−1 0	+1 +2	−6 −5	−3 −2	0 +1
Class C	+7	+4	+1 +2	+4 +5	+6 +7	+2 to +2	+1 +2	+3 +4	−4 to +2	−1 to +2	+2 +3
Class D	+8 & up	+5 & up	+3 & up	+6 & up	+8 & up	+3 & up	+3 & up	+5 & up	+3 & up	+3 & up	+4 & up

Class A (large flaws)
Any indication in this category shall be rejected (regardless of length).

Class B (medium flaws)
Any indication in this category having a length greater than 3/4 inch (19 mm) shall be rejected.

Class C (small flaws)
Any indication in this category having a length greater than 2 inches (51 mm) shall be rejected.

Class D (minor flaws)
Any indication in this category shall be accepted regardless of length or location in the weld.

Scanning levels	
Sound path	Above zero reference, dB
to 2-1/2 (64 mm)	14
>2-1/2 to 5 (64-127 mm)	19
>5 to 10 (127-254 mm)	29
>10 to 15 (254-381 mm)	39

Notes:

1. Class B and C flaws shall be separated by at least 2L, L being the length of the longer flaw, except that when two or more such flaws are not separated by at least 2L, but the combined length of flaws and their separation distance is equal to or less than the maximum allowable length under the provisions of Class B or C, the flaw shall be considered a single acceptable flaw.
2. Class B and C flaws shall not begin at a distance less than 2L from weld ends carrying primary tensile stress, L being the flaw length.
3. Flaws detected at "scanning level" in the root face area of complete penetration double groove weld joints shall be evaluated using an indication rating 4 dB more sensitive when such welds are designated as "tension welds" on the drawing (subtract 4 dB from the indication rating "d").
4. Electroslag or electrogas welds - Flaws detected at "Scanning Level" which exceed 2 in. (51mm) in length shall be suspected as being piping porosity and shall be further evaluated with radiography.
5. To obtain equivalent values in cm multiply listed values by 2.54.

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Section 4

ISOLATION OF SPECIFIC FACTORS WHICH RESULT IN WELD FLAWS

4.1 Classification

The majority of weld imperfections can be categorized according to their nature (Table 16). The imperfections so listed should not be considered as rigorously assigned to a given category, because they may have secondary attributes in other categories.

From AWS's terminology, a weld defect is: "A discontinuity which by nature or accumulated effect render a part or product unable to meet minimum applicable acceptance standards or specifications". This term designates rejectability, i.e., a defect is defined as a weld imperfection that is harmful to the service performance of the part. On the other hand, a flaw may be defined as a weld imperfection which is not harmful to the service performance of the part.

Planar types such as cracks, laminations, lack of fusion and lack of penetration generally have a more pronounced stress amplification effect than three dimensional imperfections. Therefore, not only should we characterize imperfections by their nature, but we should always consider their:

1. Size
2. Acuity
3. Orientation
4. Location

Those imperfections lying in a direction such that the acting stresses tend to cause opening are more detrimental than those at other angles to

Table 16

Characterization of Weld Imperfections

A. Metallurgical Causes:

1. Hot Cracks
2. Cold or Delayed Cracks
3. Microfissures
4. Delamination of Base Plate
5. Reheat or Stress-relief Cracks
6. Microstructure Alternation of Heat-affected Zone (HAZ)
7. Weld Metal and HAZ Segregation

B. Geometrical Causes:

1. Poor Fit-Up . Mismatch
2. Under Cut and Cavity
3. Overlap
4. Excessive Reinforcement
5. Concavity and Convexity
6. Nature of Weld Dressing

C. Weld Process and Procedures Related Causes:

1. Lack of Fusion
2. Incomplete Penetration
3. Slag Inclusions
4. Porosity
5. Arc Strikes
6. Burn Through
7. Entrapped Weld Spatter
8. Insert Ring Lack of Fusion
9. Backing Ring Lack of Penetration
10. Tungsten Inclusions in GTA Welds
11. Copper Inclusions
12. Oxide Films
13. Surface Irregularity Ripples

D. Design Related Causes:

1. Stress Concentration in General
2. Weld Joint Type

E. Residual Stress Causes:

1. Constraint
2. Repair Welding

the applied stress. It is also recognized that surface or near surface imperfections may be more detrimental than a similarly shaped interior imperfection.

4.2 Weld Imperfections Common to Bridge Structural Elements

4.2.1 Metallurgical Causes

4.2.1.1. Hot Cracks

Hot cracks occur at temperatures near the solidus at which the last molten metal freezes, and are usually attributed to the presence of a liquid or mushy phase in the grain boundaries. This mushy phase is unable to withstand the weld shrinkage and thermal stresses imposed on it, thereby resulting in separation along these boundaries as cracks (Figure 10).

In steel, sulfur is one of the principle causes of hot cracks in welds. Excess sulfur, present as FeS, forms a low-melting eutectic which forms a film in grain boundaries and reduces cohesion. Manganese will tie up the sulfur as MnS and prevent the formation of the harmful FeS film.

The main metallurgical factors are brittle grain boundaries and intergranular phases that have low melting points. Mechanical factors include joint design, shape and thickness of the weldment, and size and shape of the weld bead.

Hot cracks are generally intergranular and interdendritic. In the base metal, hot cracks caused by welding are usually found in the heat-affected zone. With time, these cracks may propagate through the weld.

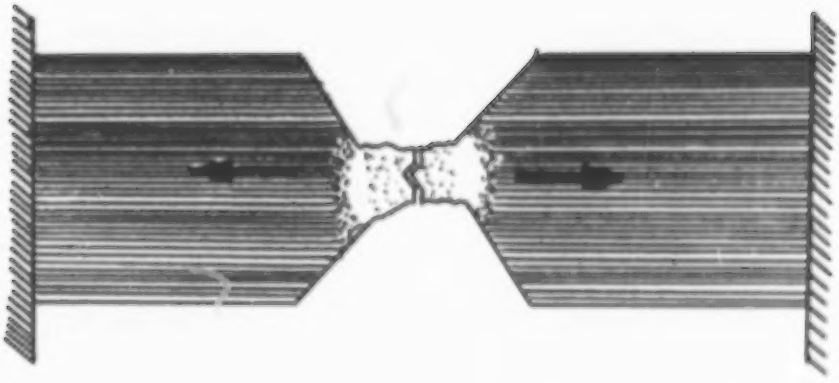

Figure 10a - Butt Weld With Center Line Crack

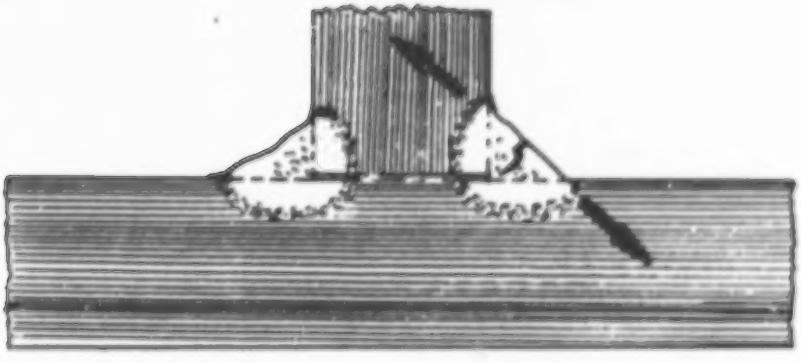

Figure 10b - Fillet Weld With Center Line Crack

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Origins (24): (1). The imposition of excess transverse strains in restrained welds. This condition may result from welds having a depth-to-width ratio in excess of 2:1, a situation which is most likely to occur with those welding processes which have high current densities, such as submerged-arc and CO_2 welding. (2). With high content of elements, C, P, S. High dilution from the parent plate and preheat may also be contributory factors. (3). Contaminants remaining on the parent metal, such as cutting oils. (4). Fillet weld having gross concavity and large gaps between the component parts.

Detection: Surface: visual examination, either direct or with the aid of low power magnification; dye penetrant and magnetic particle testing. Internal: Ultrasonic and radiographic examination.

Correction: Cut off defective weld length plus 0.2-inches (5mm) beyond visible end of crack and reweld.

Prevention: (1). Plan welding procedures to reduce thermally induced strains. (2). Adjust weld energy parameters to produce depth-to-width ratios of about 1:1. (3). Avoid parent steels containing more than 0.06% total of sulfur and phosphorus. (4). Clean off all traces of cutting oils. (5). Control joint fitup to reduce gaps. (6). Reduce weld metal dilution.

4.2.1.2 Cold Cracks

In steels, cold cracking occurs at temperatures below 400°F(200°C), more often at or near room temperature. It may occur hours, or even days, after the weldment has equalized in temperature and is free of thermal stresses.

Cold cracking in steel is primarily associated with the formation of a brittle phase (martensite) or with hydrogen embrittlement. As the austenite in the base metal cools, its capacity for holding dissolved hydrogen diminishes. When the supersaturated austenite transforms to martensite near room temperature, cracking may occur in the martensite.

Cold cracks are usually transgranular. They generally start in the base-metal heat-affected zone just under the weld head, unless the weld metal is harder than the base metal. These cracks run parallel to the weld and are called either toe cracks or underbead cracks. The former occur near the cap pass, the latter near the root of the weld (Figure 11).

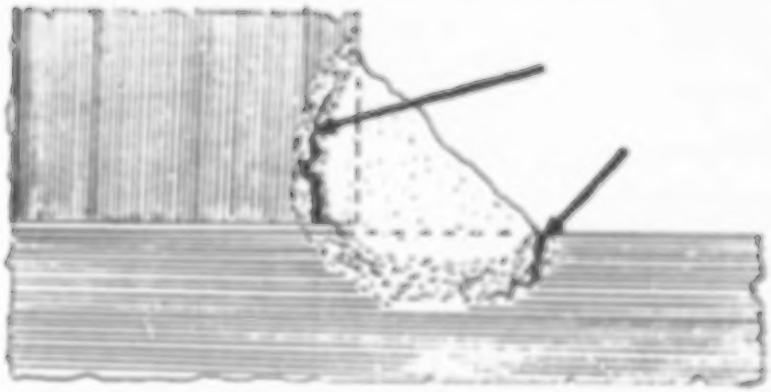

Figure 11 Hydrogen-Induced (HAZ) Cracking

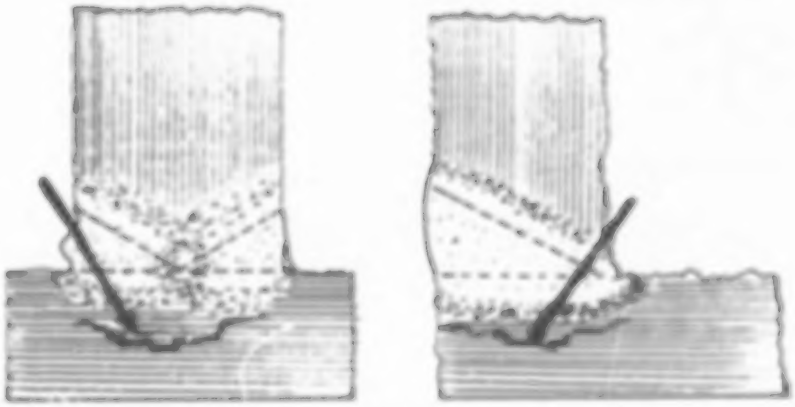

Figure 12 Lamellar Tearing
(see page 78, 1st paragraph)

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Origins: (1). The diffusion of hydrogen from the weld metal into hardened HAZs. The hydrogen may come from moisture in or on the welding consumables or on the joint faces, and from other contaminants such as paint, oil. The probability of cracking increases with increase in section thickness and increases in carbon equivalent. Large root gaps, low heat input, and the absence of, or insufficient, preheat may also be contributory factors. (2). Unforeseen handling stresses after welding but before stress relieving. (3). Tack welds of inadequate size.

Detection: Surface: visual examination, dye penetrant and magnetic particle testing. Internal: ultrasonic examination.

Correction: Short isolated lengths of crack, where accessible from the surface, can be cut out and repaired by welding. Extensive and/or sub-surface cracking may require the replacement of the affected parts.

Prevention: (1). Welding consumables must be of the hydrogen-controlled type and used under controlled conditions. (2). Electrodes must be dried in accordance with the manufacturer's instructions. (3). Preheat, where applicable, must be such that the full section of the joint is raised to the required temperature. (4). Heat input requirements must be carefully maintained even for erection tack welds. (5). Parent metal composition must be within the limits which are consistent with the welding procedure used.

4.2.1.3 Microfissures

Microfissures are cracks so small that they cannot be seen with magnification of less than ten diameters. Microfissures are probably present in most base and weld metals. They may be caused by either hot or cold cracking.

4.2.1.4 Delamination of base plate (Lamellar Tearing)

Lamellar tearing is defined as cracks in steel plates parallel to the plate surface which originate adjacent to welds. They are not, in general, associated with the HAZ and display a characteristic step formation (Figure 12).

Origins: (1). Poor ductility in the through-thickness direction of the rolled plate. The low ductility is caused by thin zones of nonmetallic inclusion lying beneath and parallel to the plate surface. (2). Incorrect joint design. (3). Thermally induced strain in restrained joints resulting in high stresses in the through thickness of the plate.

Detection: Surface: stepped appearance is its particular characteristic. Internal: ultrasonic inspection.

Correction: Small areas which have experienced lamellar tearing can be completely cut out and replaced with weld metal. When extensive lamellar tearing is discovered, complete substitution of all defective material is often the only safe solution.

Prevention: Joints should be designed so that weld contraction imposes the minimum of strain in the through thickness direction. Where the joint design cannot be altered, techniques which place high ductility weld metal in the direction of the strain are used.

4.2.2 Geometrical Causes

4.2.2.1 Poor Fit-up, Misalignment (Figure 13).

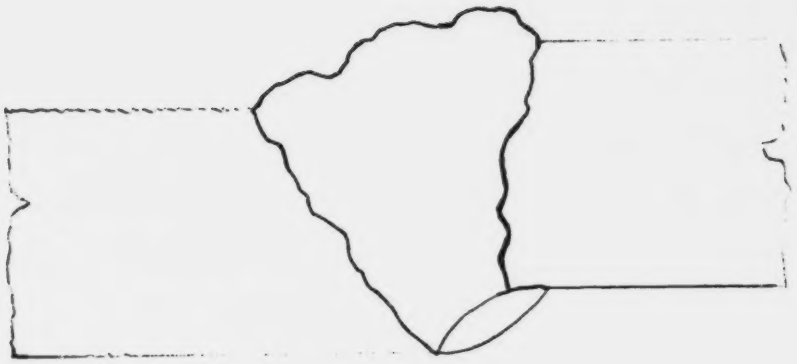

Figure 13 The Nonalignment of Two Abutting Edges in a Butt Joint

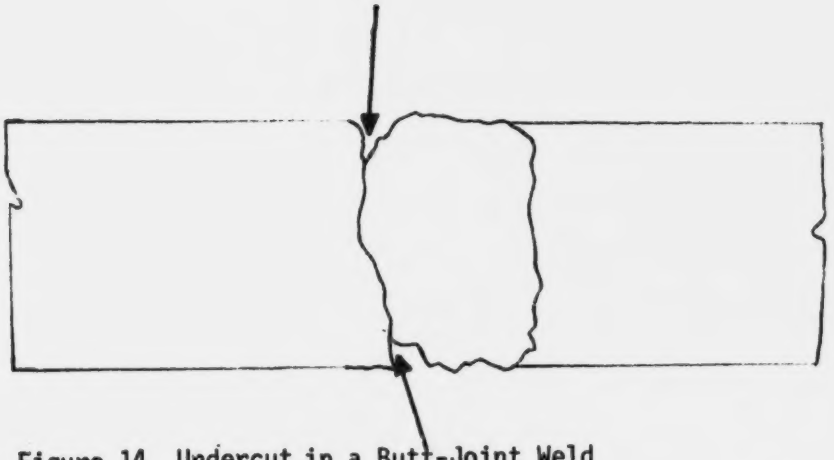

Figure 14 Undercut in a Butt-Joint Weld

(see page 80, 5th paragraph)

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Origins: Inaccuracies in assembly procedures; distortion from other welds; and excessive "out of flatness" in hot rolled plates or section.

Detection: Visual inspection, use a welding gauge to determine the degree of misalignment between the working pieces before welding.

Correction: Correction will be carried out only if the misalignment is beyond specified limits. Note that corrective treatment may impose high restraint and, therefore, introduce a risk of cracking.

Prevention: Adequate checking of alignment prior to welding, coupled with the use of clamps, wedges, or other devices. Preweld checking of the rolled section accuracy with possible correction before assembly of the joint.

4.2.2.2 Undercut

Undercut is a groove melted into the base metal adjacent to the toe or root of a weld and left unfilled by weld metal. Undercut is a linear surface imperfection that may vary in depth, width, and sharpness at the root of the notch (Figure 14).

Origins: (1). The melting away of the top edge by use of a high welding current. (2). Attempting to produce a horizontal/vertical fillet with a leg length greater than .35-inches (9.0mm). (3). Excessive weaving; incorrect weaving. (4). Incorrect welding condition, e.g., travel speed too slow.

Detection: Visual examination, depth measured by welding gauge.

Correction: For shallow undercut, the groove can be blended into the plate by grinding. For deep undercut, the defect should be repaired by welding, perhaps after grinding.

Prevention: Weld with the work in the flat position. Reducing weaving by changing to multi-run techniques. Change shielding gas to one which

produces better metal wetting, e.g. add from 0.5 to 5.0% oxygen to argon in MIG-welding. Terminate welds so that they do not finish at a free edge.

4.2.2.3 Overlap

An imperfection at the toe or root of a weld caused by metal flowing onto the surface of the parent metal without fusing to it (Figure 15).

Origins: (1). Poor manipulation of the electrode in manual metal-arc welding; excessive weaving. (2). Energy input too low to fuse the parent metal adequately. (3). Travel speed too low when associated with high energy input. (4). Work not positioned correctly.

Detection: Visual examination, supplemented by dye-penetrating testing.

Correction: By cutting back to sound metal and, if necessary, rewelding.

Prevention: Adjustments to electrode manipulation to ensure fusion of the parent metal. Limiting the size of horizontal/vertical fillet welds to .35-inches (9.0mm) length.

4.2.3 Weld Process and Procedures Related Causes

4.2.3.1 Lack of Fusion

Lack of fusion or incomplete fusion occurs when some portion of the weld metal is not completely fused with the base metal, or previously deposited weld metal. This commonly occurs when the arc is not hot enough to melt the base metal, or if the weld metal runs ahead of the arc and cold casts into the joint without melting and fusing to the base metal. Incomplete

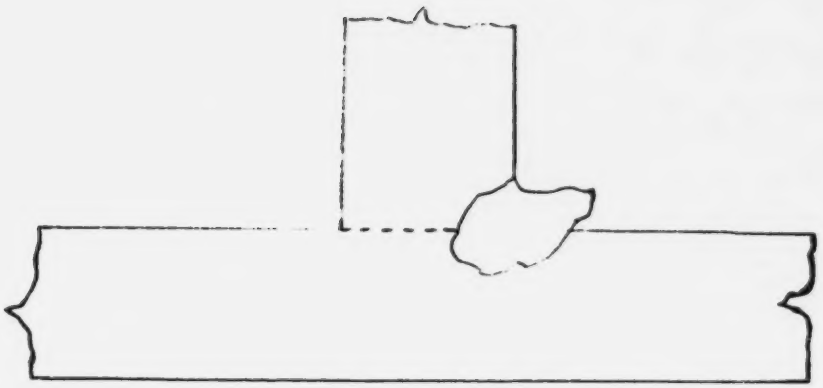

Figure 15 - Overlap

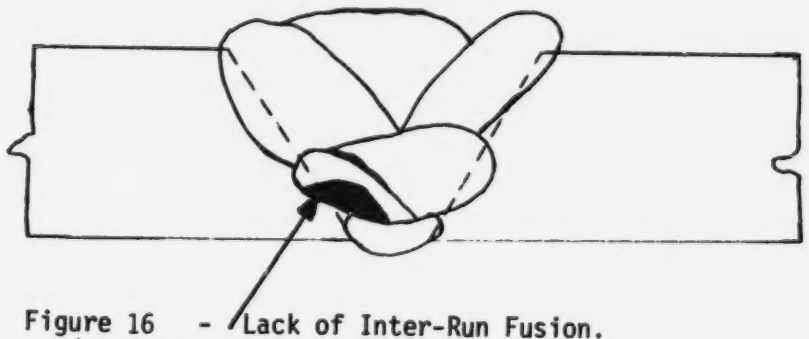

Figure 16 - Lack of Inter-Run Fusion.
(see page 83, 1st paragraph)

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fusion is commonly found between the weld metal and base metal, along the sides of the first or second pass in a joint where the joint is tightest, and where it is most difficult to properly direct the arc. (Figure 16).

Origins: Incorrect welding conditions, e.g., arc input energy too low; travel speed too fast and incorrect electrode angle.

Detection: Surface: If the lack of inter-run fusion reaches the weld surface, magnetic and dye penetrant testing can be used. Internal: By ultrasonic examination, but the results may be difficult to interpret.

Correction: The defective area should be cut-out and rewelded.

Prevention: The welding procedure for complete fusion should be corrected. Excessive inductances in GMAW or CO_2 welding should be reduced even at the expense of increased spatter.

4.2.3.2 Incomplete Penetration

Incomplete penetration, lack of penetration, or inadequate joint penetration occurs when the joint penetration is less than that specified. In full-penetration welds, the joint is to be completely fused. Presence of an unfused joint surface at the root of the joint constitutes a lack of penetration defect. In partial penetration joints, the groove is to be completely fused. If the weld has not penetrated to the bottom of the groove, the void constitutes a lack of penetration defect. (Figure 17).

4.2.3.3 Slag Inclusions

Slag inclusions are nonmetallic, solid material entrapped between weld-metal passes, or between the weld and the base metal. Slag is produced in the welding process either by decomposition of welding fluxes,

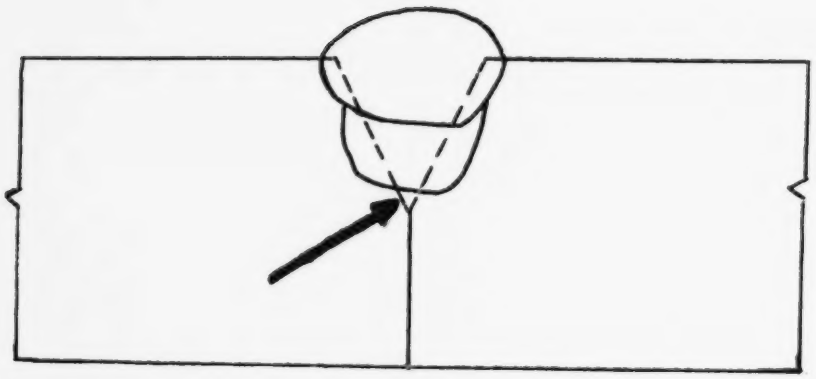

Figure 17 - Lack of Penetration

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or by reactions of alloying elements in the weld with oxygen. It may occur as short particles similar to porosity or as long bands similar to incomplete fusion.

Origins: (1). Loss of slag control because of poor manipulative techniques, or slag flooding in advance of the arc because of work position. (2). Incomplete removal of solidified slag from the underlying runs of a multipass weld. (3). The poor bead profile produced by some electrodes may give rise to slag inclusions in multi-run welds.

Detection: Internal: ultrasonic or radiographic examination.

Correction: The defective area should be cut-out and rewelded.

Prevention: The work should be positioned to prevent loss of slag control, or the electrode (or flux) changed to improve control. Inter-run slag removal should be improved, and if the weld surface is likely to cause slag traps the irregularities should be dressed smooth.

4.2.3.4 Porosity

Porosity is defined as the existence of gas pockets or voids in the weld metal. It is usually caused by evolution of gas either from the molten weld metal, or from foreign substances entrapped in the weld metal. If these gases cannot escape from the weld puddle before solidification, the weld will be porous. Porosity is also believed to occur from mechanical entrapment of atmospheric gases. Pores are usually spherical in shape, although they may occur as nonspherical pockets along grain boundaries or as elongated, tubular voids called "worm holes". Large, isolated gas pockets -- usually irregular in shape -- are sometimes called "blowholes". Porosity may occur anywhere in the weld metal. Most welds will contain some amount of porosity, and depending

on the size and frequency of pores, they may or may not constitute a defect.

Origins: Worm holes are caused by the progressive entrapment of gas between the solidifying metal crystals (dendrites), producing the characteristic elongated pores of circular cross-section. These elongated pores often appear as a "herringbone" array on a radiograph. The gas may come from gross surface contamination. Worm holes can also originate from plate laminations if these terminate in the weld metal.

Detection: Surface: visual examination. Internal: radiographic and ultrasonic examination.

Correction: The defective area should be cut out and repaired, using the original welding procedure after a contamination check has been made. If the worm holes have been caused by a lamination, it must first be cut back and sealed off with a weld.

Prevention: Preweld cleaning must be carried out. The location of laminations should be established and the work prepared so that welds do not pass through the laminated zone. If laminations are exposed on a prepared joint face, they should be sealed first by local welding.

4.2.3.5 Arc Strikes

Random areas of fused metal where the electrode, the holder, or current clamp have accidentally touched the work and produced a short duration arc.

Origins: (1). Poor access to the work. (2). Missing insulation on electrode holder or torch. (3). Failure to provide an insulated

resting place for the electrode holder or torch when not in use.

(4). Loose current return clamp.

Detection: Visual examination: magnetic or dye penetrant testing may reveal that the arc strikes are cracked.

Correction: By light grinding the arc strike areas so that any incipient cracking is removed.

Prevention: Improve access, perhaps by modifying the assembly sequence. Use only insulated holders and torches, and institute a regular inspection scheme for electrode holders and torches. Current return clamps should also be regularly maintained. Welders should be made aware that arc strikes can lead to dangerous cracking.

4.2.3.6 Burn Through

Burn through is a void or open hole extending through the bottom of the weld joint. It is caused by local overheating of a spot in the bottom of the joint during deposition of the first or second weld pass. When this spot becomes molten, the metal runs out of the joint leaving a hole or window in the bottom of the joint. Alternately, the molten metal may not run out of the joint, leaving a hole. It may merely sag and form icicle like structures on the bottom of the joint; this condition is sometimes called "melt through" or "icicles". In the case of joints welded against a backing strip, complete melting through in a local area is also called melt through. In this case the metal sags to form icicles or it stays essentially in place. Burn through or melt through can occur only in welds deposited from one side of the plate.

4.2.3.7 Entrapped Weld Spatter

This weld imperfection consists of small droplets of electrode material which have been thrown clear of the weld, and may or may not be fused to the parent plate. Some spatter is always produced by open-arc, consumable electrode welding processes.

Origins: (1). Incorrect welding conditions, e.g., excessive energy input; excessive arc length, and magnetic arc blow. (2). Contaminated welding consumables such as damp manual metal-arc welding electrodes.

Detection: Visual examination.

Correction: Remove by scraping or light grinding.

Prevention: Reduce energy input; use a shorter arc length; reposition current return to reduce magnetic arc blow, or change to alternating current. Increase inductance in GMA and CO_2 welding, but not at the expense of full fusion.

4.3 CONSEQUENCES OF WELD IMPERFECTIONS

Some examples of the consequences of weld discontinuities (imperfections) will be given below:

4.3.1 Cracks

Owing to their sharpness, cracks are detrimental to the fatigue behavior of weldments, and are amongst the most severe defects. It has been observed that micro-cracks are the cause for high-yield strength steels exhibiting the same S-N curve as mild steels. In most cases, cracks are considered unacceptable and need to be repaired.

4.3.2 Undercut

Undercut can reduce the cross sectional area and cause a stress concentration. When fatigue testing weldments under alternating loads, fatigue cracks very often initiate from undercut and propagate through the base metal. High tensile steels with a high-yield strength/tensile strength ratio are very sensitive to this notch effect; therefore; undercut must be removed when these steels are used under fatigue loading.

4.3.3 Lack of Fusion and Incomplete Penetration

These two discontinuities may cause brittle failure under either static or dynamic loading. If these discontinuities are aligned perpendicular to the applied alternating stress, the influence is maximized.

Long-life fatigue strength of low carbon steel weldments can be severely diminished by joint penetration discontinuities.

4.3.4 Slag Inclusions

Slag inclusions reduce the tensile strength in proportion to their projected area and have a small effect on yield strength. The way slag inclusions act as stress raisers is very similar to that of porosity. They are considered to have less influence on fatigue strength if the weld reinforcement is not removed, and if they are buried and not at the surface.

Harrison [25] has made a critical assessment of the influence of slag inclusions on high cycle fatigue of butt welds, and summarized the maximum permissible lengths of slag inclusions for four different welding conditions and five quality levels (Table 17).

Quality	Rutile Welds As-Welded	Low-Hydrogen Welds As-Welded	Stress Relieved Welds	As-Welded High Restraint or Cross Welds
V	0.0	0.0	0.0	0.00
W	1.5	5.0	5.0	0.75
X	10.0	25.0	NM	4.00
Y	NM	NM	NM	25.00
Z	NM	NM	NM	NM

Units of lengths in mm. To obtain equivalent units in inches multiply listed values by .04.

Table 17 MAXIMUM PERMISSIBLE LENGTH OF SLAG INCLUSIONS.

Quality Level	V	W	X	Y	Z
Max. Allowable Volume of Porosity, %	0	3	8	20	20

Table 18 ALLOWABLE PERCENTAGE OF POROSITY BY VOLUME FOR THE VARIOUS QUALITY LEVELS. (see page 91, 1st paragraph)

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4.3.5 Porosity

In the scattered, unaligned form, porosity has been shown to have the least influence on the static properties, if present in amounts less than 5% or in some cases up to 7%.[26] By comparing data from fatigue tests of butt welded steel specimens containing different levels of porosity, Harrison[27] proposed allowable percentages of porosity (by volume) for each quality level as shown in Table 18.

Porosity which breaks the surface is more injurious than pores within the inner structure of the material. For fillet welds, porosity rarely affects the fatigue properties unless it is located at the root (Figure 18).

4.3.6 Crater Pipes

Crater pipes with cracks can act as initiation sites for fatigue cracks and brittle fracture.

4.3.7 Excessive Overfill

Figure 19 shows the influence of the overfill angle on fatigue strength of A36 mild steel. A combination of excessive overfill and undercut may be one of the most dangerous combinations of weld defects with regard to fatigue strength.

4.4 APPARENT FREQUENCY

As an example of the typical frequency of fusion welding defects resulting from high-quality, field-welding practice, the following example is offered:

During the fabrication of a 3-mile (4.8Km) pipeline, fourteen types of welding defects were identified by X-ray examination of the girth welds. The inspection of the pipeline was controlled by the American Petroleum Institute Standard No. 1104. Table 19 and Figure 20 show the weighted frequencies of the various defects.

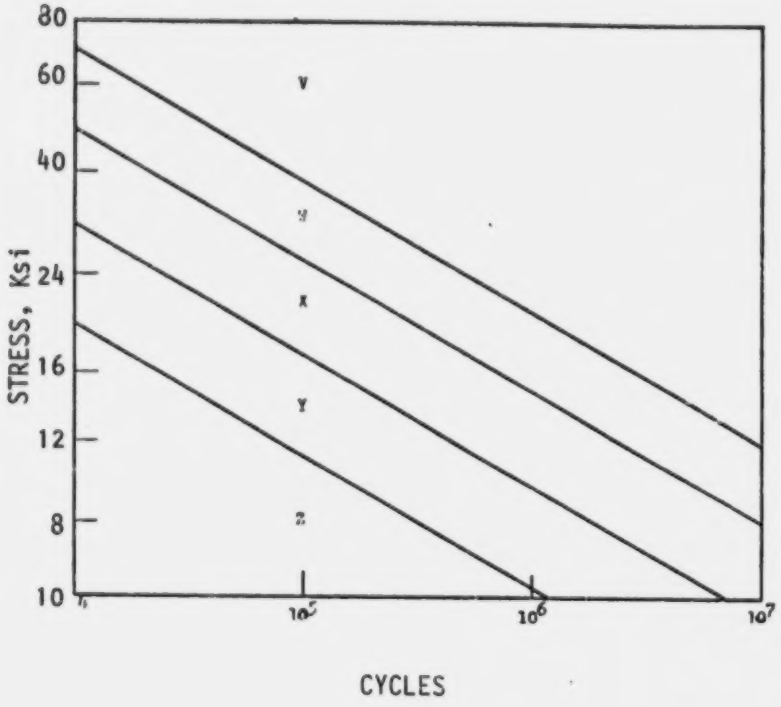

Figure 18 - Fatigue Test Results for Porosity Levels from 0% to 20+% . To obtain Equivalent Values in MPa Multiply Vertical Axis Numbers by 6.9. See Table 18 for Definitions of Areas Identified by Letters.

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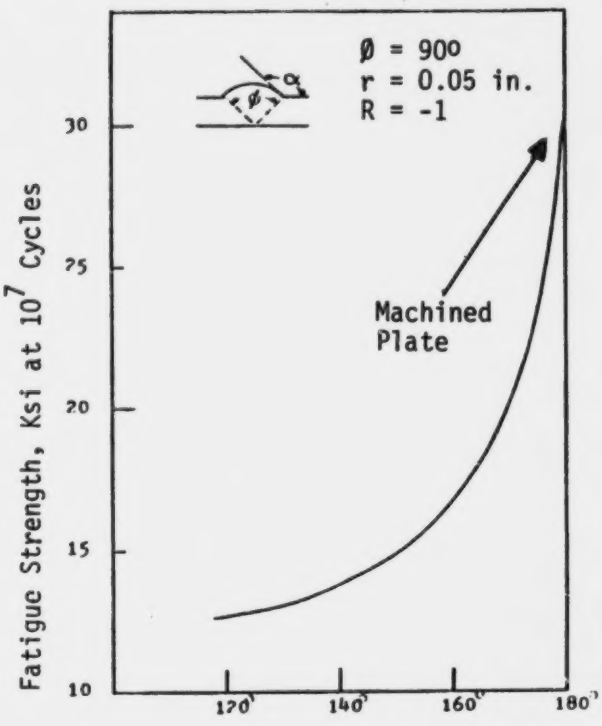

Figure 19 Influence of Overfill Angle on Fatigue Strength for A36 Mild Steel. To Obtain Equivalent Values in MPa Multiply Vertical Axis Numbers by 6.9.

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Table 19

Weighted Frequencies of Defects in Pipeline Weld

Defect	Type	No. of Identifications			Weighted Frequency (%)		
		Out of Code	In Code	Total	Out of Code	In Code	Total
A	Cluster Porosity	1	2	3	0.4	0.7	1.1
B	Burn Through	2	2	4	0.7	0.7	1.4
C	Porosity	4	8	12	1.5	2.9	4.4
D	Cracks	20		20	7.3		7.3
E	Outside Undercut	19	8	27	2.3	7.6	9.9
F	Slag Inclusions	30	27	57	11.0	10.9	20.9
G	Incomplete Penetration	47	18	65	17.3	6.6	23.9
H	Gas Pocket	18	10	28	6.6	3.6	10.2
I	Hollow Bead	16	8	24	5.9	2.9	8.8
J	Concave Bead	13	2	15	4.8	0.7	5.5
K	Non Fusion	9	1	10	3.3	0.4	3.7
L	Shrinkage	2	1	3	0.7	0.4	1.1
M	Internal Cavity	1	2	3	0.4	0.7	1.1
N	Arc Burn	2	0	2	0.7		0.7

Total No. of Identifications 273

Weighted Frequency = No. of Identifications/Total No. of Identifications

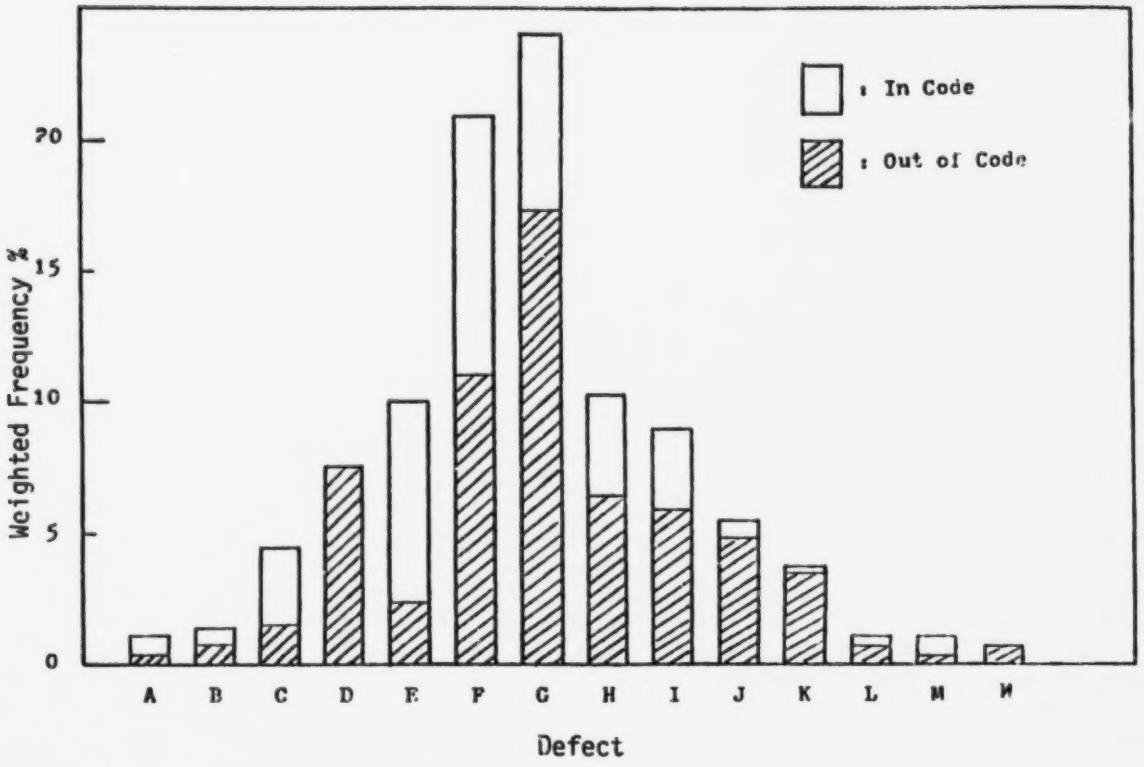

Figure 20 -- Frequency of Various Defects in 3 Mile Pipeline. See Table 19 for Definitions of Defects Identified by Letters.

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4.5 MECHANICS MODELS FOR PREDICTING THE SEVERITY OF WELD DEFECTS

4.5.1 For Static Loading

The stress intensity factor K is a function of the stress (S), the crack length (c) and a geometry factor (Y):

$$K = YS(c)^{\frac{1}{2}}$$

Brittle fracture is considered to result when $K = K_{Ic}$ or K_c, the fracture toughness.

Maddox has shown for elliptical flaws, Y may be considered to be the superposition of several effects =

$$Y = \frac{M_S M_t M_k}{\phi_o}$$

where: M_S is a magnification factor which corrects for the presence of a free surface, M_t is a magnification factor which corrects for finite plate width $(\sec \pi a/2w)^{\frac{1}{2}}$, M_k is a magnification factor which corrects for the stress concentration of weld discontinuity in question, and ϕ_o is a correction factor for crack shape.

4.5.2 For Fatigue Loading

Lawrence [28] proposed an approach to rating the severity of an internal defect in fatigue applications, by comparing the expected N_t due to fatigue failure at the defect with the expected N_t due to weld discontinuities necessarily present (weld toes). A defect causing failure before the weld toe must be considered as detrimental; whereas, one which could not cause failure would be innocuous. The relative severity of an internal defect will depend upon the scale of the weldment (t), upon the absolute

size of the defect (c), upon material properties (S_u), upon the shape of the weldment (α^{Toe}, α^{Defect}) and upon the type of loading (axial or bending). For example, for axial loading the ratio ($K^{LOP}_{fmax} - 1$) for the LOP in the cruciform weld to that of the weld toe ($K^{Toe}_{fmas} - 1$) is:

$$(K^{LOP}_{fmas} - 1)/(K^{Toe}_{fmax} - 1) = (\alpha^{LOP}_A / \alpha^{Toe}_A)\ (S^{LOP}_u / S^{Toe}_u)\ (c/t)^{\frac{1}{2}}$$

where: $K^{LOP}_{fmas} - 1 = 0.053\ \alpha S^{LOP}_u\ c^{\frac{1}{2}}$ (in steel), $K^{Toe}_{fmax} - 1 = 0.053\ \alpha S^{Toe}_u\ t^{\frac{1}{2}}$ (in steel).

Thus, for axial loading, an LOP defect in a cruciform weldment (shown in Figure 21, for example) should become more serious a fatigue initiation site (at long $N_t \simeq N_1$) than the weld toe when the ratio exceeds 1.

In general, the relative severity of an internal defect to the weld toe may be expressed as:

$$(S^{Defect}_u / S^{Toe}_u)\ (c/t)^{\frac{1}{2}} \qquad \{\alpha^{Defect}_A + (\Delta S_B / \Delta S_A)\ \alpha^{Defect}_B\} /$$

$$\{\alpha^{Toe}_A + (\Delta S_B / \Delta S_A)\ \alpha^{Toe}_B\}$$

where: A denotes axial loading, B denotes bending. The defect will become detrimental when the value of the above expression is greater than 1.

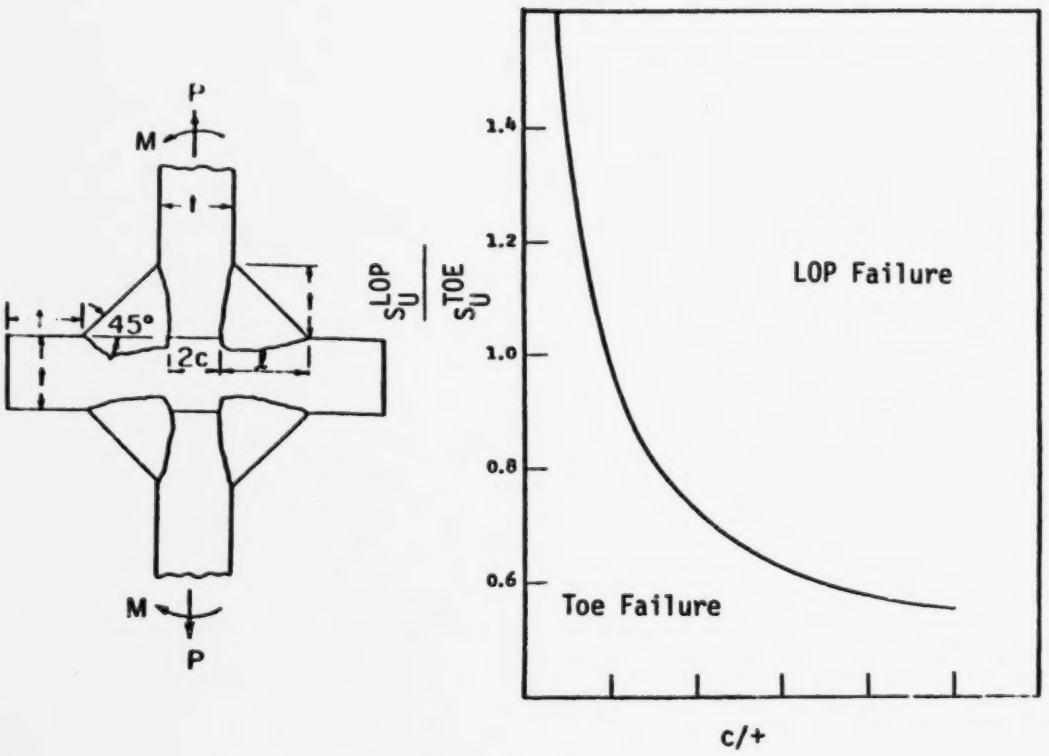

Figure 21 - An LOP Defect Axially Loaded in a Cruciform Weldment

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Section 5

RECENT RESEARCH ON METALLURGICAL ASPECTS OF BRIDGE WELDING

Ideally a weldment represents an exact continuation of properties from the adjoining base metal plates. However, this is generally not the case because of solidification events in the weld metal as well as the localized heat input into the base metal. Consequently, the weldment represents a discontinuity of various properties with properties differing from those of the undistributed base plate. Considerable effort is presently being expended to understand, characterize, and quantify particular aspects of the weldment as a metallurgical discontinuity in terms of properties and structure.

5.1 General Weld Considerations

The welding process can be characterized as a melting process in which a highly superheated liquid metal (filler metal) is allowed to solidify in its container. In fact, it is necessary to allow fusion of the container surface to provide the desired volume continuity from base metal across the weld to the base metal on the other side. Otherwise, undercuts or "lack of fusion" defects result with the familiar detrimental effects on weld properties.

These high superheating temperatures required to achieve reliable fusion of filler metal and base plate have a number of frequently undesirable side effects well known in the metal casting industry but occurring highly localized in a weldment, e.g.

Gas pick-up by the liquid metal

Segregation and gas occlusion during and after solidification due to changing solubility

Directional solidification

Solidification shrinkage

Liquid filler metal dilution by incipient melting of the base metal surface.

Microstructural development on cooling and due to solid state transformations and reaction products

Density changes in cooling and transformation resulting in intermittent and residual micro-and macro stresses.

To control all these events for optimum weld properties, base metal, filler metal, and flux must be optimally matched and coupled with a suitable heat input. The degree of progress achieved in this optimization process is reflected somewhat in the increasingly complex welding code requirements outlined earlier. Still, failures of welded structures are abundant indicating the need for both greater attention to detail (e.g., base metal, filler, flux selection, flaw detection) and more tolerant welding conditions in the field. (e.g., less constraint on heat input requirements).

According to recent reports[29, 30] submerged arc welding is still the more economical, and technically the most reliable method for quality welds. It appears to be the preferred welding process for automatic welding if the geometry is not overly complicated. The preferred method for the root pass is the Tungsten-Inert-Gas (TIG) process, especially, if it is not possible to gouge and reinforce from the opposite side.

5.1.1 Weldability

The primary purpose of welding is to join two base materials without loss of base material properties across the junction. The welding process, however, involves temperatures above the melting point complete with chemical reactions and reaction products from all available elements participating to various degrees depending on temperature. As a consequence, localized flaws arise which reduce the structural integrity and fracture toughness of the bridge as a whole.

Any metal can be welded. The important point in production is the indefinite term "readily". If the welding procedure is such that even a fraction of the total carbon goes into solution, the rapidly cooled heat affected zone will be hard and brittle. Preheating, post heating and controlled cooling after welding therefore become important. If the welding, preheating and postheating are done correctly, a satisfactory junction results. However, more things can go wrong than what is mentioned above when welding steel. If preheating and postheating are not controlled, cracking of the heat affected zone will be chronic. If the welding procedures are not controlled and the base metal is heated above its melting point too long, a drastic change in structure may occur on solidification.

A frequently used formula to assess the weldability of a steel based on composition is

$$CE(\%) = C + \frac{Mn}{4} + \frac{Si}{4}$$

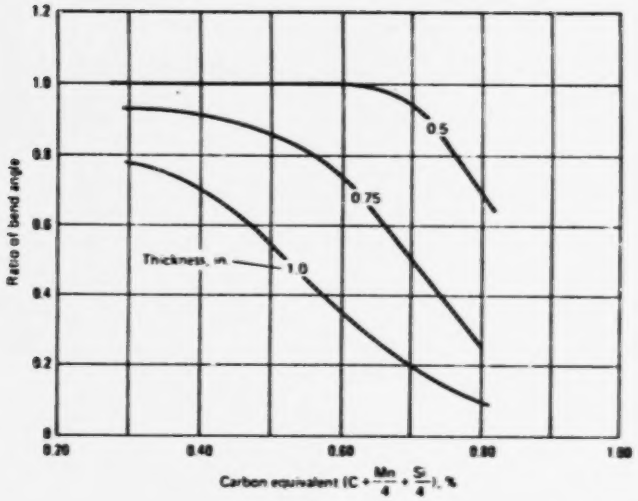
A high value indicates lower weldability for a given section size[31] (Figure 22).

Research has established that the sensitivity of a steel to hydrogen induced cracking is more closely related to the hardness of the weld heat affected zone (HAZ). This hardness is, in turn, related to the base plate tensile strength (Figure 23) and to the base plate chemistry characterized by carbon equivalent formula:

$$CE\ (\%) = C + \frac{Mn}{6} + \frac{Cr+Mo+V}{5} + \frac{Ni+Cu}{15}$$

(ITW Carbon Equivalent Ref. 32)

For all steels studied by Delvechio et.al.[33] the maximum underbead hardness which correlated best with P_{cm}, is the Ito-Bessyo cold cracking parameter [34]



A high value of the ratio indicates high weldability. Derived from data on notched bend test by Stout and Doty, Weldability of Steels, Welding Research Council, 1953. To obtain equivalent values in cm multiply thickness numbers by 2.54.

Figure 22 Ratio (Welded to Unwelded) of Bend Angle For Normalized Steel Plate (31)

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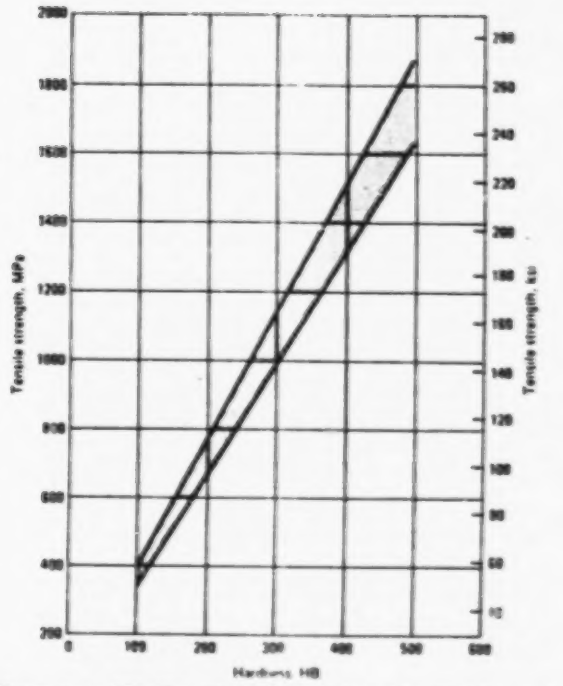

Range up to 300 HB is applicable to hot finished steel discussed here. (SAE Handbook).

Figure 23 Relation Between Hardness and Tensile Strength of Steel

(Ref. ASM Handbook Committee, "Metals Handbook Ninth Edition", Volume 1 - Properties and Selection: Iron and Steels, 1978)

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$$P_{cm} = C + \frac{Si}{30} + \frac{Mn + Cu + Cr}{20}$$

$$+ \frac{Ni}{60} + \frac{Mo}{15} + \frac{V}{10} + 5B$$

The potential for close chemical control, therefore, is obvious, but it must be balanced against the tolerance of the base metal to be joined taking into account variations in weld practice.

5.1.2 Weld Repair

Fabrication by welding is usually based on specifications that describe in detail a particular set of welding conditions. If these are not maintained, insufficient, or wrongly applied, then acceptance criteria set by design factors may not be met. If the weldment is rejected, it may not be scrapped, but usually an assessment is made whether it could be repaired.

According to latest estimates[35], weld repair of major proportions is required for 25% of existing bridges over the next 10 years. Facing this problem research was initiated in weld repair at the Welding Institute[36,37], and specifications for repair of heavy welded sections were established by the New York State Department of Transportation and by the American Association of State Highway and Transportation Officials (AASHTO). The following weld flaws, in particular, have been subject to consideration for these standardized repair procedures:[35]

- Cracking
  - e.g., hydrogen or delayed cracking
- Undercut
- Lack of fusion
- Failure to back gouge

Some characteristic cracks are illustrated in Figure 24. These are primarily due to excessive shrinkage and thermal expansion stresses. They

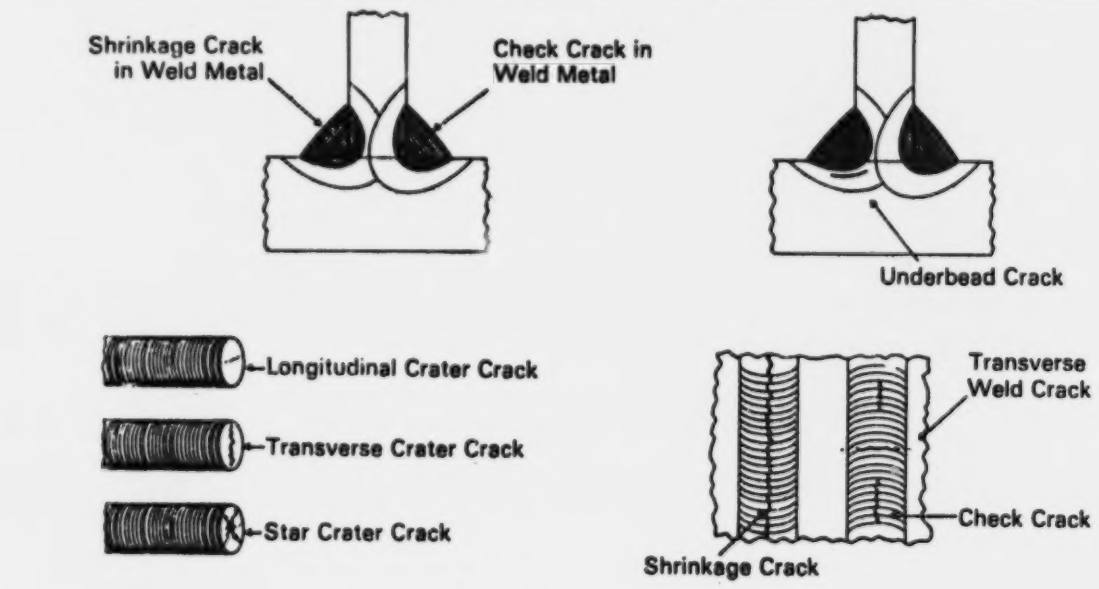

Figure 24 Characteristic Cracks

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are the result of lack of control over suitable pre- and post heating as well as improper choice of weld metal and heat input. Hydrogen cracking is usually the result of using unsuitable welding rods, not low in hydrogen content and/or of the wrong composition. Undercut, lack of fusion, and failure to back-gouge can be attributed to improper weld preparation, inattentive welding, and improper process and equipment selection.[35]

5.2 Weld Materials

The characteristics of the A36,, A514 and A588 type steels can be summarized in the following way:

Type A36 is the most common grade of structural steel used in bridges and buildings and for other general structural purposes. Its microstructure consists of regions of ferrite and pearlite. Type A514 structural steel is a high-strength, low-alloy, quenched and tempered steel used in such applications as welded bridges, buildings, and pressure vessels. This steel shows usually quenched and tempered martensitic microstructures. ASTM type A588 steels are high strength, low-alloy structural steels generally used where weight savings and added corrosion resistance are important.

5.2.1 Filler Metal

Nickel bearing filler metals still appear to be most suitable for acceptable fracture toughness values for a wide variety of joint designs, welding gas, and heat inputs.

North et.al.[39] have examined notch toughness of weldments deoxidized with Al, Rare Earths (RE), Mn, and Mg whose effects are quite cooling rate dependent. Zirconium additions still are better if the oxygen content is kept below 200 ppm (11.7 grains/gallon). Vanadium tends to promote the desirable acicular ferrite and results in improved toughness. At low cooling rates, RE are beneficial. Aluminum tends to be most suitable for plain carbon-manganese

deposits such as expected in the weldments of A36 type steels.

In view of their important and distinctly different compositional effects, the amount of carbon and the presence of Silicon, Manganese, Molybdenum, and Nickel, suggestions have been made in France [40] to incorporate this information in wire electrode and flux designations used in submerged arc welding. Additional efforts have been made to incorporate symbols for finish, heat treatment, and dimensional characteristics as well as weight.

5.2.2 Flux

Entrekin[41] recently reported on results from studies made on the effect of flux basicity on the microstructure, hardenability, and impact properties of welds in typical micro-alloyed steel. Double vee butt welds in a columbium-bearing steel were produced by submerged arc welding with a carbon-manganese filler wire with additions of molybdenum, titanium, and boron and two fluxes with basicity indices of 1.37 and 4.97.

Since cleavage cracks run preferentially through the proeutectoid ferrite grains during fracture, limitation of the amount of this microconstituent is desirable. The higher basicity flux reduced this constituent. In addition, molybdenum, titanium, and boron added to the flux proved equally beneficial.

As the amount of proeutectoid ferrite in the weld is reduced, the toughness will increase. The added contributions of the above trace elements to hardenability tend to help in suppressing this detrimental constituent, and do so most effectively when applied together with a basic flux.

5.2.3 Materials Trends

Traditionally, the construction industry - including bridge building - has used basic low carbon, mild steel. For many years the most commonly used structural steel for bridges was A283 D steel with no compositional

specification and a minimum of 33 ksi (228 MPa) yield strength. Since its carbon level was generally around 0.3%, it had fair weldability.

Increasingly design trends shifted from tensile to yield strength, as design criterion, resulting in increasing use of A 36 type steel as of about 1960. This steel achieves generally 40,000 psi(276 MPa) yield strength, but with the added cost of some manganese alloying. Still it had a more favorable price to yield strength ratio than the earlier common steels. The framework of the 100 story John Hancock Tower in Chicago, for instance, is completely built of A 36 type steel. The use of this steel has, however, diminished in favor of high-strength, low alloy steels.

With welding becoming increasingly prominent in the fabrication of structural steel, as in bridges, alloying elements other than carbon and manganese were considered. This resulted in alloy steels such as the A 514 types. Better weldability and low impact-transition temperatures were obtained initially by lowering the carbon content, resulting in a more favorable manganese/carbon ratio.

This effect also avoided acicular transformation products in hot rolled or normalized sections. Yet, inspite of the alloy additions, lower price to yield strength ratios were achieved.

Bridges are structures in which not only yield strength but deflection plays a major role in design considerations. Since deflection is generally a function of the E-modulus with little dependence on alloy contents for low alloy steels, there is a limit to the extent to which high strength levels in a bridge steel will be economical. A 588 type high-strength low alloy steels have somewhat lower strength, but compensate with gains in weldability and corrosion resistance in exposed atmospheres. Most improvements in the HSLA type steels are presently directed toward improved impact

properties or improved formability.[42]

5.2.4 Effect of Residual Tin

Tin is expected to increase in American Steel by 1985 to 0.016% from the present 0.008% level. In anticipation of this increase, Stephenson[43] assessed the effect on various mechanical properties of A36 and A514 type steels. The effects on these common grades appear negligible. For example, an increase in 0.01% Sn increased the transition temperature from $0^{o}F(-17^{o}C)$ to $4^{o}F.$ $(-15.6^{o}C)$. Similarly small effects were noted on shelf energy, reduction of area, and elongation.

5.3 Process Control

As pointed out earlier, weldability of a material is frequently associated with the relative values of CE and P_{cm} parameters which are based on chemistry only. The higher these parameters, the more critical become additional parameters such as heat input, filler, and flux selection as well as pre and post heat treatment. Non-compliance with the established welding codes will then result in a number of undesirable phenomena subject to considerable attention in recent research efforts.

5.3.1 Distortion

To combat buckling problems, the British Institute of Structural Steels in London[44] has limited post-welding distortion tolerances for stiffened plates in compression to

$$\text{maximum imperfection} < \frac{\text{gauge length}}{100}$$

An intensive investigation by the British Dept. of Transportation of critical flatness of plate elements of grade 43 steel has resulted in the data of Figure 25 and consequently more stringent requirements.

W_A- Measured Longitudinally over Length "b"
W_B- Measured Longitudinally over Length "2b"
W_C- Measured Transversely over Length "b"

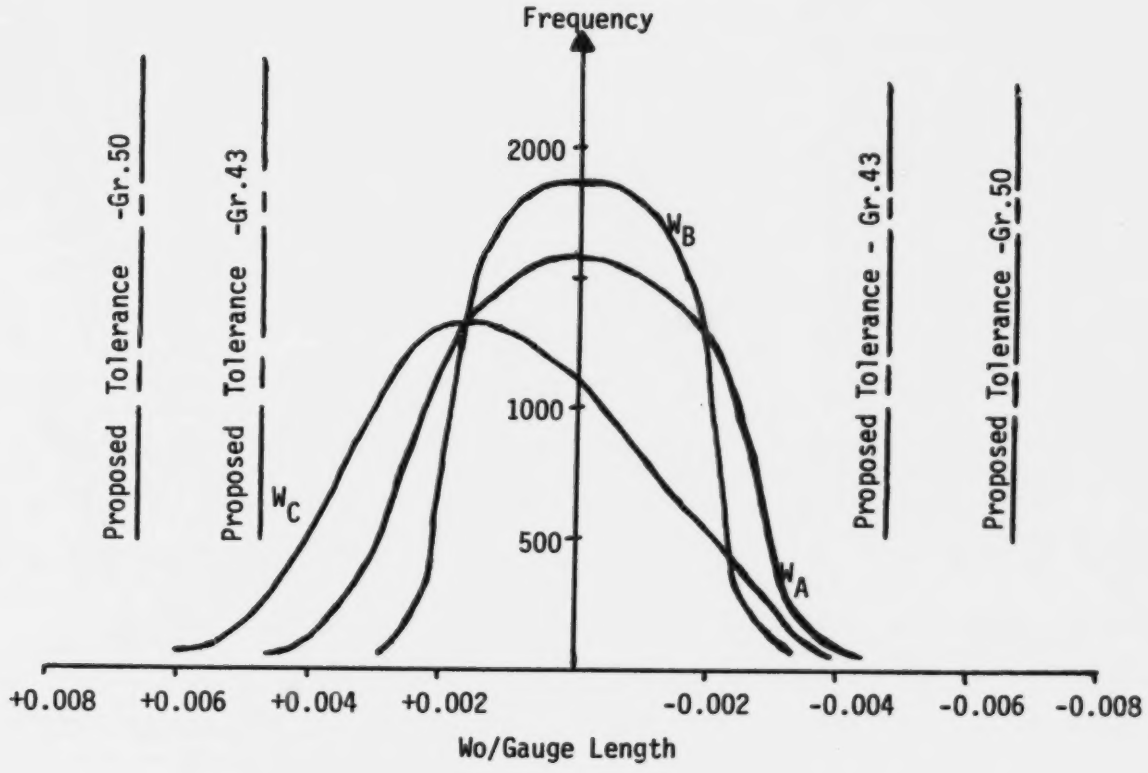

Figure 25 STATISTICAL DISTRIBUTIONS OF IMPERFECTIONS

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$$\text{maximum imperfection} < \frac{\text{gauge length(b)}}{200}$$

In view of this new data, values of

$$\frac{b}{200} \quad , \quad \frac{b}{165} \quad , \text{ and } \quad \frac{b}{150}$$

have been proposed for grades 43, 50 and 55 respectively based on a balanced approach to what flatness can be practically maintained and what buckling requirement must be met.[45]

5.3.2 Residual Stresses

Most measurements of residual stresses near weldments have been made on butt-welded flat plates, and residual stress distributions are reasonably well characterized in this case. Stresses transverse to the weld are generally small compared to those parallel to the weld, unless the outside edges of the plate are constrained during welding. The stress parallel to the weld direction is tensile in a region that may extend to several times the plate thickness on either side of the weld centerline. Close to the weld, stresses approaching yield may be expected.[46]

Residual stresses in weldments are due primarily to nonuniform cooling resulting in thermomechanical deformations above the yield point at some locations and in the elastic range at others. They may originate during the welding process, and/or post-weld surface and heat treatments.[47] Analytical models for residual stresses near weldments are available[48-54] and show that peak stresses occur adjacent to the fusion line slightly modified by various types of surface treatments.

Attempts to relate residual stresses to weldability have been made by numerous investigators.[55,56] Results, however, need to be interpreted for A36, A514 and A588 type steels.

5.3.3 Hydrogen

The effect of hydrogen is generally considered as detrimental in steels[57] and is frequently associated with the presence of impurities which catalyze the trapping, or diffusivity of the hydrogen atoms available. Other results suggest that the familiar embrittlement is associated with the interaction of hydrogen and impurities during drastic straining or severe hydrogen charging conditions. It was found that in the presence of hydrogen, charging grain boundaries without impurities do not lead to fissures but impurities without grain boundaries do.

Hydrogen trapping appears to be a dominant cause of hydrogen related problems. Carbide interfaces represent strong traps[58] while substitutional elements such as Ti provide weaker traps. A rough guideline would be the expected stability of the binary hydride for the hydrogen getter* atom in question. Strong hydride formers segregate to grain boundaries and free surfaces such as cracks and voids attract hydrogen accumulation.[59,60] The presence of atoms with larger molar volumes on the other hand, such as oxygen will block the entry of hydrogen.

The presence of hydrogen appears to have little effect on the yield stress but reduces ductility and the critical stress intensity for crack propagation.[61,62,63] The effects are accentuated for higher strength steels where most commonly hydrogen induces failure at lower stress or intensity levels characterized by reduced plasticity and plastic zone size, and the corresponding fractography is that of quasi cleavage or brittle cleavage.[64] In these steels, delayed failure under sustained loads is observed at stresses at levels of σ_y, the yield stress, or less[57,65,66]. Additions of Ti tend to provide weak traps for hydrogen preventing it from reaching more damaging interfaces.[67,68] Other impurities, S, P, and

* A getter atom adsorbs impurities that are inherent in base materials.

metalloids tend to hydrogen embrittle as does carbon near open circuit conditions.[69] Manganese is detrimental when segregated to grain boundaries or inclusion interfaces. Microstructurally the susceptibility to hydrogen degradation increases in this order: lower bainite, quenched and tempered martensite.[67,70,71] Fine grain size and fine carbide sizes are beneficial, as is increased cleanliness as provided by vacuum melting.[72]

Elements and heat treatments which promote temper embrittlement also promote hydrogen cracking along prior austenite grain boundaries, along grain boundaries, and in the transgranular mode.[73,74,75] Manganese promotes degradation by adsorbing to boundaries and attracting P and Si, Si acts by interacting to increase the activity of P and Si in solution and thus increase the chemical potential gradient driving diffusion of P and Si to boundaries.[74]

5.3.4 Fracture

Welding is an important method of joining single members into complex structural systems required to function under diverse loading conditions in construction engineering. Arc welding has been widely used to join various constructional alloy steels. In the arc welding process, the arc supplies the heat needed to melt the filler and base metal surfaces being joined. Even when the proper electrodes, heat input, and weld techniques are used,[76] weld deposits are rarely free of discontinuities: many types of defects may occur, including porosity, lack of fusion (LOF), lack of penetration (LOP), cracks, undercuts, inclusions, and burn-through. The effects of these defects on the mechanical properties of the joint differ and depend on such factors as size, shape, orientation, distribution, and service environment, all of which play important roles in determining the performance of a weldment.

The relative importance of these effects is discussed in Section 4. The following sections relate some recent findings in the literature.

5.4 Temper Embrittlement

This problem is primarily associated with the heat treatment of the higher alloyed steels such as the A 514 type steels. Its incidence should be minimal during welding to present code specifications if cooling rates of the weldment through the 662-1022°F (350-550°C) range are not prolonged artificially.

5.4.1 Hydrogen Embrittlement

Studies on hydrogen-assisted cracking (HAC) are numerous and so are related mechanisms. In an attempt to sort out the most likely applicable cracking mechanisms Lynch[77] interpreted evidence from fractographic examinations, indicating that HAC occurs on plastic flow rather than by decohesion of the grains at crack tips. The most plausible theory attributes hydrogen embrittlement in steels to chemi-sorption of hydrogen atoms which facilitates nucleation of dislocation at crack tips.

Consistent with the proposed theory appears to be the fact that the susceptibility of HAC of steels increases as the strength increases. Dislocations emitted do not have to move far to contribute to crack growth in higher-strength steels.

High yield strength steels are generally prone to hydrogen embrittlement. A 4-year program at Battelle Columbus Laboratory to examine the usefulness of rare earths in gettering hydrogen during welding and, thus, reduce the potential for hydrogen embrittlement in HY-130 steel welds was concluded recently.[78] The work revealed that rare earth-nickel compounds in a cored wire were most effective in gettering. To prevent oxidation of the rare earths, the cored wire was used as a cold-wire addition during submerged-arc welding. When welds were made with a moisture-laden flux, rare earth flux,

and the rare earth cored wire, there was no evidence of hydrogen cracking. Delayed transverse hydrogen cracking occurred in welds made with the damp flux alone. However, in gas metal-arc welding lowered cracking propensity was associated with reduced mechanical properties and, frequently, increased arc stability.

5.4.2 Hot Cracking

Weldable structural quality steels are customarily produced with sulfur levels of less than 0.03%. If the manganese to sulfur ratio is greater than 30 with a carbon content of less than 0.2%, the susceptibility of hot cracking is negligible.[79,80] The susceptibility is also negligible for higher carbon steels if the Mn/S ratio approaches 40. These conditions, as well as absence of tramp elements such as Zn and Sn is usually achieved in A36, A514, A588 steels indicating that hot cracking due to materials would be rare.

5.4.3 Cold Cracking

If hydrogen is kept sufficiently low most weldable constructional steels have negligible susceptibility to cold cracking. Because this cracking may occur hours or days after welding, it is also known as delayed cracking. The mechanism is complex.[81,82,83] Welding specifications of the American Welding Society[84] recognize the importance of low hydrogen electrodes and permit the use of lower preheat and interpass temperatures when such electrodes are used. Still, care must be exercised to prevent recharging of low hydrogen electrodes as seen in Figure 26 and 27.

5.4.4 Lamellar Tearing

Lamellar tearing is the brittle cracking of a steel plate along the surfaces of non-metallic inclusions, when normal tensile stresses are present[85] due to the contraction of weld metal on cooling (Figure 28).

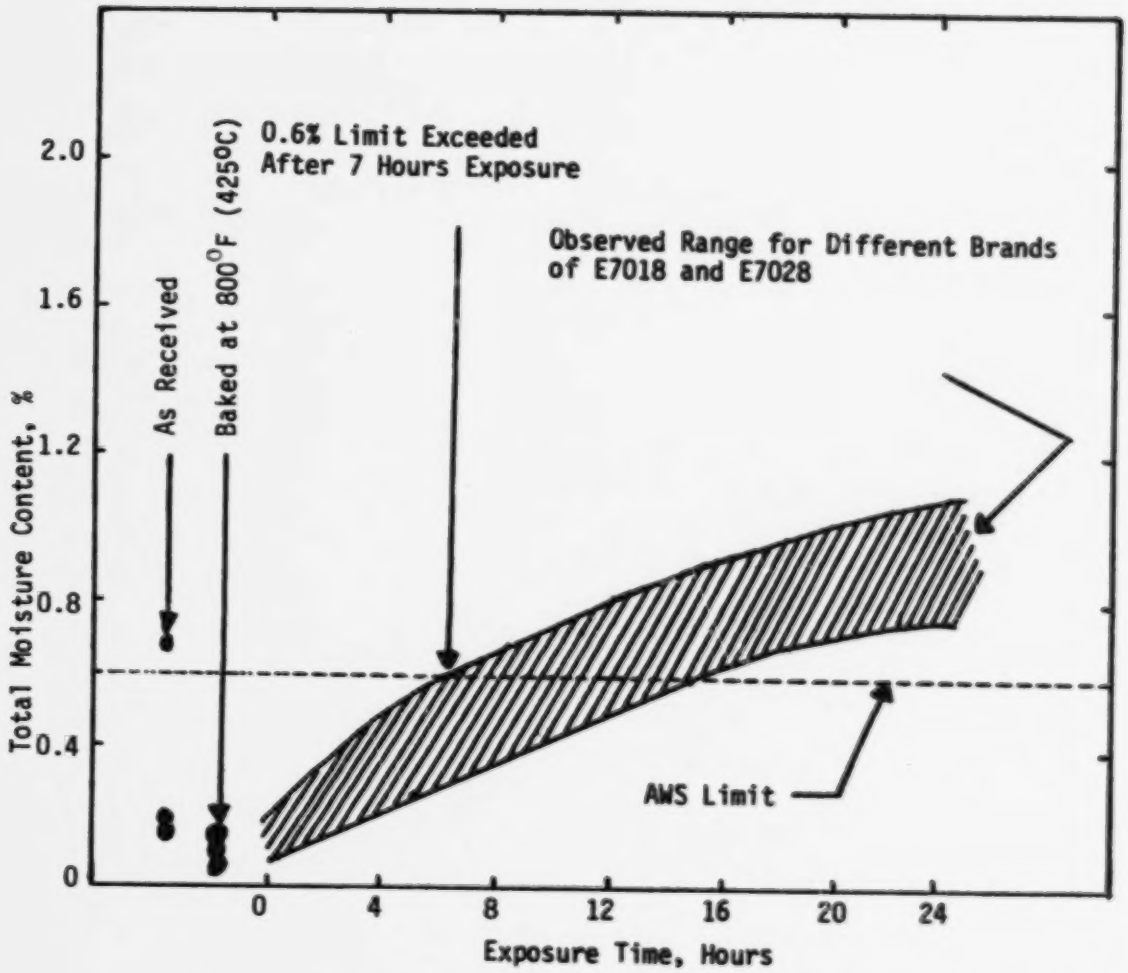

Figure 26 Moisture Content of E7018 and E7028 Electrodes Exposed at 75°F (23°C) to an atmosphere of 60% Relative Humidity. (81)

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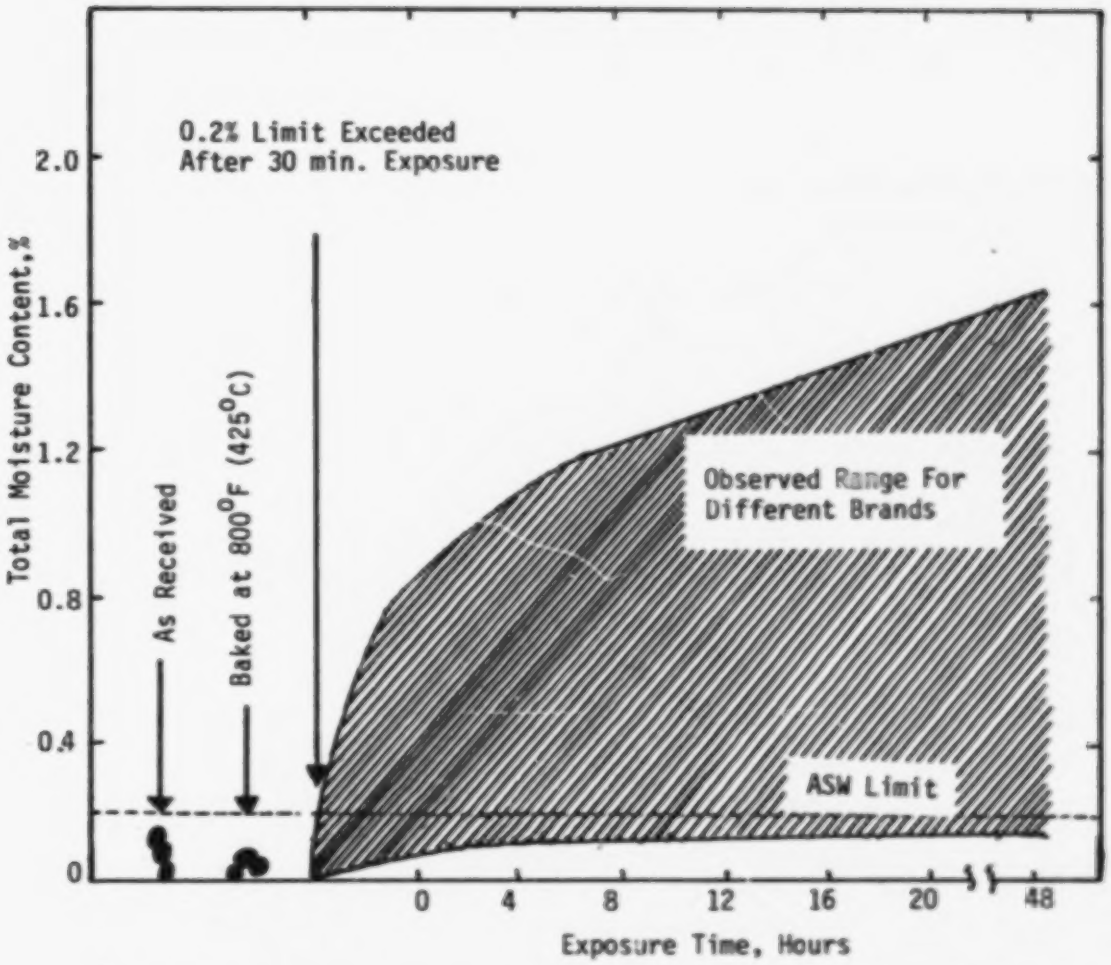

Figure 27 Moisture Content of E11018-G Electrodes at 75°F (23°C) to an Atmosphere of 60% Relative Humidity. (81)

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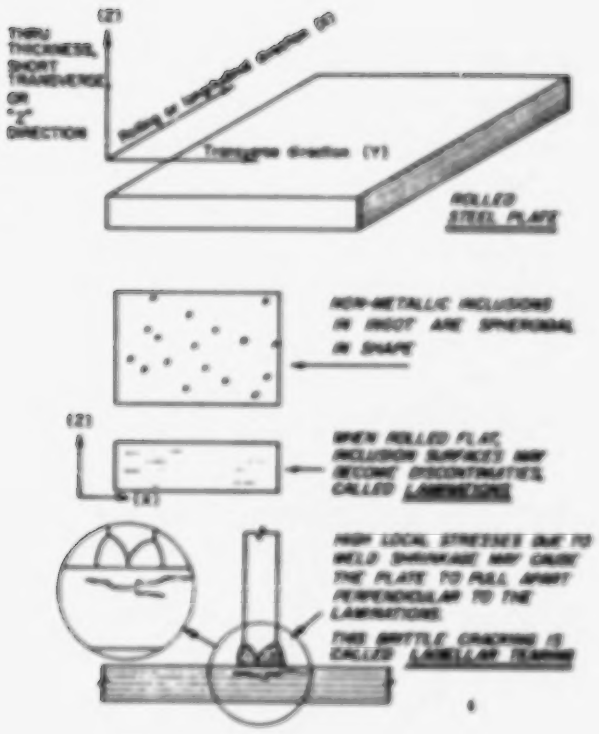
(Z)
THRU THICKNESS, SHORT TRANSVERSE, OR "Z" DIRECTION
ROLLED STEEL PLATE
NON-METALLIC INCLUSIONS IN INGOT ARE SPHEROIDAL IN SHAPE
WHEN ROLLED FLAT, INCLUSION SURFACES MAY BECOME DISCONTINUITIES, CALLED LAMINATIONS
(X)
HIGH LOCAL STRESSES DUE TO WELD SHRINKAGE MAY CAUSE THE PLATE TO PULL APART PERPENDICULAR TO THE LAMINATIONS
THIS BRITTLE CRACKING IS CALLED LAMELLAR TEARING

Figure 28 Description of Lamellar Tearing (85)

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Joints are most susceptible when thick plates, large weldments, and high internal connection restraint are all present. Table 20 illustrates the various steel making techniques to exercise inclusion shape control for improved tear resistance.

Lamellar tearing must also be considered a warning signal for the possible presence of other problems associated with the anisotropic nature of rolled plate as shown by Kato et.al.[86] In this case, experiments resulted in failures on through-thickness loads and displacements less than those predicted by conventional designs.

5.4.5 Inclusion

The effect of inclusions is to lower the fatigue life. Since most loads peak at the surface, their presence is most deleterious at surface locations.[87] Present steel making techniques address inclusion control as shown in Table 20. However, welding joints also tax the through-thickness inclusion condition as shown by the tendency toward lamellar tearing and lower evidence limits for dirtier steels. Most observations indicate that fatigue crack initiation takes place at inclusion lines. Reports to the contrary fail to see the three-dimensional nature of the distribution of favorable crack initiation points - sometimes out of view of the plane of observation under the optical microscope. The degree of coherency, local residual stress, and the imposed local stress range are the determining factors.[88]

Inclusions may affect the rate of propagation of fatigue cracks running transverse as well as parallel to inclusion orientations, hence, inclusion loaded steels perform worse in fatigue both in the longitudinal and transverse direction when compared to steels with inclusion control.

Table 20

STEELMAKING TECHNIQUES TO CONTROL AMOUNT, TYPE, AND INCLUSION SHAPE

Practice	Purpose	Inclusions
Conventional	Desulfurize	MnS Type II Al_2O_3 Clusters
Ladle Flux Practice	Desulfurize	MnS Type II & III Multiphase Clusters
Calcium Treatment	Desulfurize Shape Control	MnS Type III Duplex, Modified
Electroslag Remelting	Desulfurize Shape Control Solidification Benefits	MnS Type III Duplex, Modified

Non-metallic inclusions appear to serve as points of crack initiation in addition to increasing the rate of crack propagation in the through-thickness of steel plate. Inclusion studies correlate well with predictions of a short transverse reduction of area, a common estimator of susceptibility to lamellar tearing. So, while increased plate hardness improves fatigue strength but causes weldability problems, inclusion control can improve both fatigue strength and weldability significantly. Limitations are inclusion content, size, shape, and chemical composition which can be specified for steel subjected to high cycle fatigue loading (Table 20). Such specifications may reasonably follow guidelines set for preventing lamellar tearing as the solutions for improving fatigue strength, and preventing lamellar tearing via inclusion control appear to be naturally compatible.[89]

5.4.6 Undercuts

The resistance of the steels to undercut is determined by the degree of notch sensitivity developed in the base metal by the thermal exposure in the HAZ.[90] Fatigue tests in this study, of butt welded joints served as a basis for calculating effective stress concentration factors induced by common undercut configurations.

5.4.7 Retention of Tensile Strength, Notch Toughness

Non-heat treated and normalized low alloy steels attain adequate tensile strength in welded joints. Also, the quenched and tempered types, such as the A514 steels have adequate resistance to softening if excessive heat input is avoided. However, for those A514 grades with yield strengths above 80 ksi (552 MPa) controlled cooling rates and heat input limitations must be observed. [81]

5.4.8 Heat Affected Zone

In the successful development of weldable constructural steels,

especially the heat treated steels, consideration has been given to the retention of adequate notch toughness in the heat affected zone of the weldments. The minimum cooling rate required to provide the desired microstructures depends on the hardenability of the steel which in turn depends on the given chemical composition in the heat affected zone. Consequently,[91] specifications for all grades of the A514 and A588 type steels contain limits on the allowable heat input depending on the plate thickness.

Bernard, G.[92] defines the zone extending from the fusion line to the region with a maximum temperature of 2192°F (1200°C) as the most problematic one. This is the zone of extensive grain growth.

In a related study, Monoshkovet.et.al.[93] found that resistance to and orientation of ductile failure in butt-welded joints depends strongly on the geometry of the reinforcement. The use of energy criteria on the other hand in determining the weak region and locations of failure in the weld-adjacent zone appeared to be not suitable.

According to Pellini,[94] weld properties of the HAZ may be characterized in a fashion similar to the base metal using the familiar Drop-Weight Tear (DWT) and Dynamic Tear (DT) tests. The author suggests that the protective geometry of the HAZ and generally only moderate embrittlement there, make fractures running in the HAZ only a rarity. However, even a moderately embrittled HAZ may initiate fracture instabilities ahead of a transversely located weld crack or flaw that juts from the weld into the HAZ.

It is these particular types of flaws and cracks, therefore, which force the attention on the generally weak link of the weld system and make HAZ characteristics with respect to fracture toughness so important.

Weldability tests related to the properties of the HAZ have been known for some time. The majority of weldability tests consists of modified Charpy V-notch tests. A V-notch is placed perpendicular to the weld running the entire width of the test plate. Several test plates are made in this fashion. The plates are then heated or cooled to specific temperatures, and fractured by slow loading the plates on the surface directly opposite the notch. Since the HAZ is the weakest area of the weld, failure will initiate here. Therefore the tests can be related to the HAZ properties of the weld. The design of Pellini [94] in Figure 29 shows the tip of the machined groove extending into the HAZ. The degree of bend angle on fracture after slow loading is plotted as a function of temperature and results in a transition type temperature. The degree of HAZ embrittlement is then related to the performance of a similarly tested plate without weld and HAZ. Useful in classifying successful and non-successful welding procedures for low and intermediate strength steels, the data is nevertheless only qualitative because no analytical interpretation to structural performance has evolved.

With increasing strength levels, the weld metal becomes the more difficult aspect of the problem of weldability of the steel. The HAZ of high strength metals also poses difficult metallurgical problems because of the complexity of the heat treatments developed in the welding cycles. Low alloy steels are more vulnerable in this respect because high alloying promotes the development of optimum metallurgical structures. Thus, weld HAZ for HSLA steels may be seriously degraded by off-optimum welding procedures. An example of a welding procedure which could degrade the HAZ, would be heat input into the weld controlled by the amperage level of the welding machine. If the

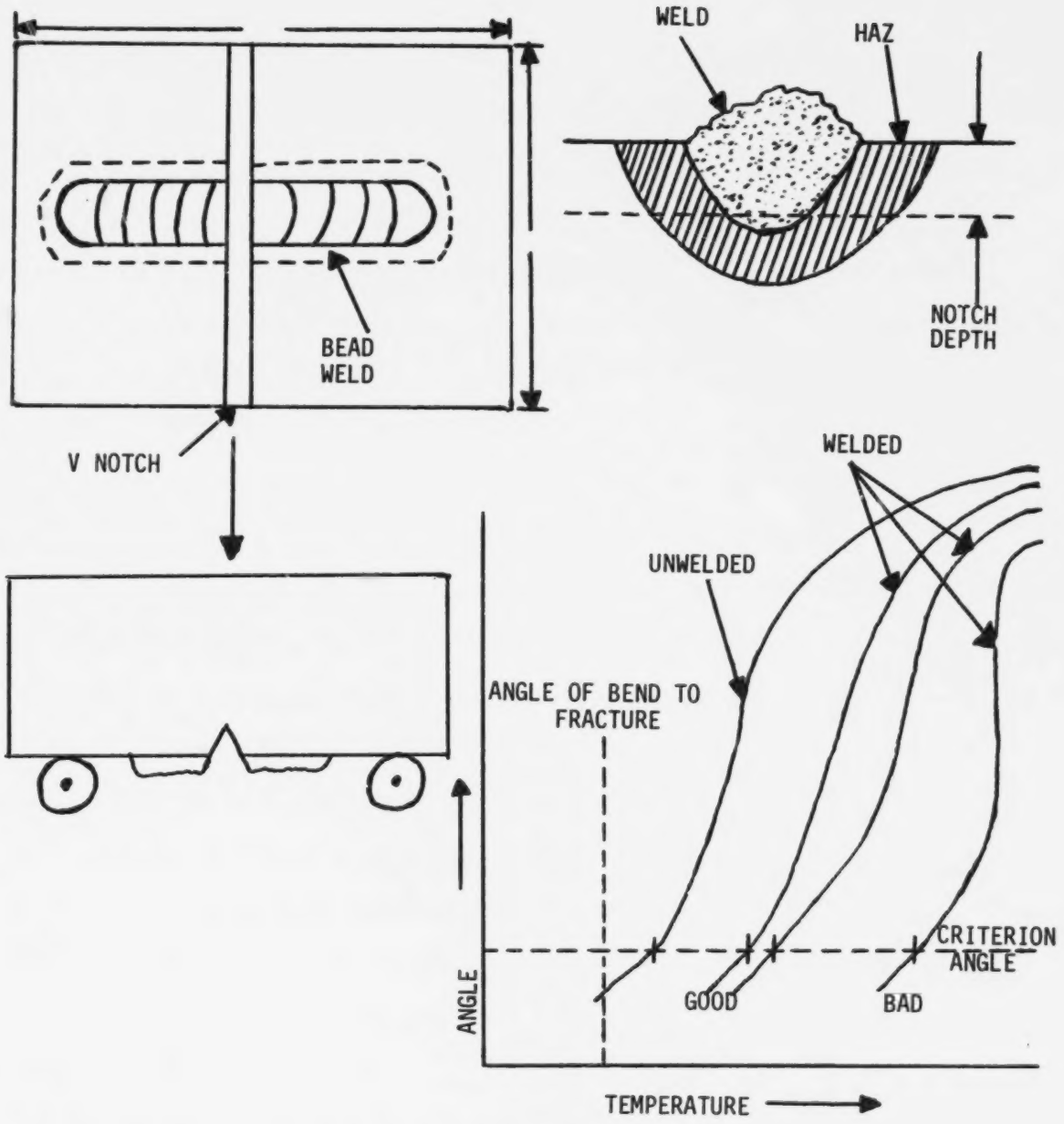

Figure 29 Features Of Weldability Tests for Assessing The Heat Affected Zone (HAZ) Characteristics For Steels Of Low And Intermediate Strength Levels (94)

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heat input is high excessive grain growth will occur in the HAZ. If the heat input is too low there will not be sufficient homogeneous mixing of the base metal and weld metal in the HAZ. Both cases will not produce optimum metallurgical structures in the HAZ which will be critical at high strength levels. Also, weld wire thickness, weld wire feed rate, welding travel speed and other welding parameters if not optimally set can degrade the HAZ. Figure 30 shows the strong relation between weld metal property and optimal welding procedures.

For characterization of HAZ properties in high strength steels the Navy uses a cylindrical bulge test featuring cracks located in the HAZ region (Figure 31). These tests are conducted after the base metal and weld characteristics are resolved to be adequate by means of the DT test. The welding procedures are then qualified with respect to the HAZ by observation that the explosion test performance is equal to that of the center or top bulges shown in Figure 31.

Grain size, cementite, and precipitates affect the notch toughness in the HAZ of high strength low alloy weldable steels. The poorest notch toughness

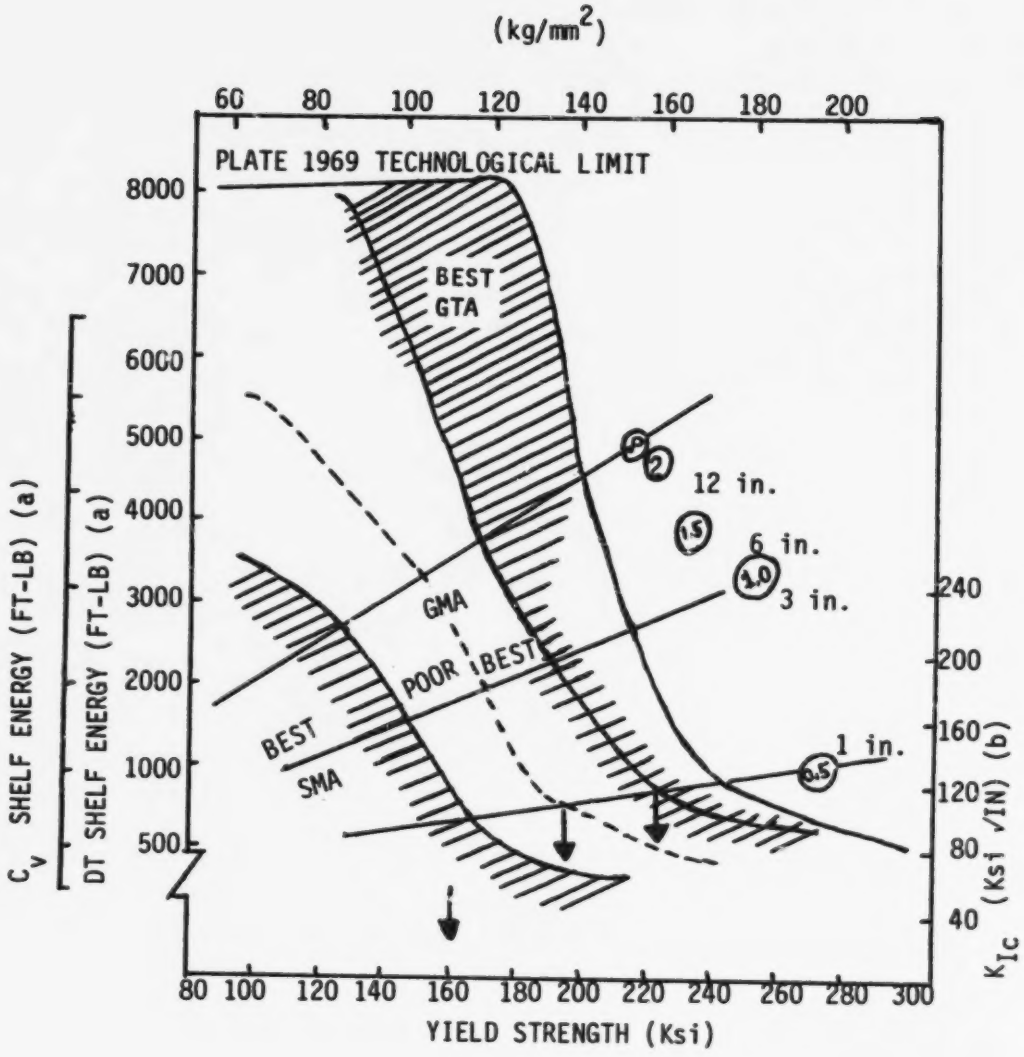

Figure 30 Zoning Of The RAD Which Defines Weld Metal Factors Of First Order Importance To Strength Transition Features. The Strength Transition Corridors Relate To The Weld Metal Deposited By Stick Metal Arc (SMA), Inert Gas Shielded Tungsten Arc (GTA). The Bold Arrows Indicate The Strength Level Of Transition To The 0.5 Ratio Value(94)

(a) To obtain equivalent units in Joules multiply above values by 1.36.

(b) To obtain equivalent units in MPa √cm multiply above values by 10.97.

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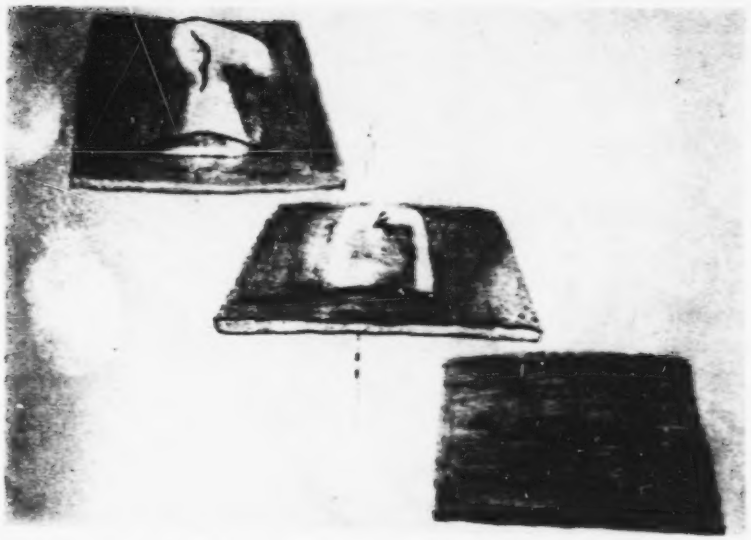

Figure 31 CYLINDRICAL BULGE EXPLOSION TEST; Series Illustrating the Structural Significance of Temperature and Strength Transitions from High to Low Positions of the Three-Dimensional Surface. (94)

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is found in the affected region heated between 1382-1742°F (750-950°C) while improved toughness was obtained at those locations where the temperature maximum exceeded 2102°F (1150°C).

Kikuta[95] et.al., have studied the notch toughness of the HAZ and related it to the quantity of cementite precipitated and the effective austenite grain size on cooling. According to their results, the amount of cementite precipitation is reduced on faster cooling from locations with peak temperatures up to 2462°F (1350°C).

Japanese work[96] has shown that the HAZ toughness of high strength weldable structural steels is improved primarily by changing a purely martensitic HAZ to a martensite - lower bainite microstructure via controlled cooling. The presence of upper bainite structures signals a marked degradation in HAZ toughness.

Schmidtmann et.al.[97] simulated HAZ effects under submerged arc conditions and found an expected decrease in formability, a grain coarsening effect, and higher hardness all of which were restored to base metal levels by the heat treatment action of subsequent passes. Fracture initiation was related to the existence of a ferritic-pearlitic tempered and normalized microstructure whereas martensitic-bainitic formations appeared to be more crack resistant. Surprisingly, the influence of grain size was deemed negligible.

5.5 Design Aspects

Product safety, a basic ingredient of product quality, has always been of considerable concern to manufacturers of components in bridge, pressure vessel, aircraft, automobile, and military structures. Wear, corrosion, and fatigue to various proportions are constantly active in altering external dimensions and internal material structures, so that final fracture must be

anticipated regardless of initial strength conditions. Initial life time predictions were based on Wohler type curves or wear and general corrosion type data. However, more recent studies are turning their attention to an analysis of the actual fracture event, its dynamics, and the possibility of controlling events actually leading up to it.

5.5.1 Fracture Control Programs

From the work Pellini[94] at the Naval Research Lab., there emerged a highly practical tool to characterize a system with respect to toughness based on flaw-size, stress, and temperature relationships. The diagram is shown in Figure 32 and requires as input the NDT temperature for a particular steel. All known cases of structural failures by instable fracture are said to have been analyzed to conform to the limits predicted by the FAD.

As the strength increases, a gradual lowering of the shelf temperatures of steel takes place. This effect has been summarized by Pellini[94] and is illustrated in the form of so-called Ratio-Analysis-Diagrams (RAD) in Figures 33 to 35. These diagrams translate information of K_{IC} and yield strength into shelf energies and, in a sense, therefore form a complementary characterization of the fracture toughness of bridge steels here under consideration.

In summary, the relations found by the work at the Naval Research Laboratories allow the estimation of the fracture toughness properties

Shelf energy

Flaw size

once data is available on K_{IC} and NDT temperatures.

Thickness Effects

It is known that the mechanical constraint associated with thickness inhibits plastic flow in the neighborhood of a flaw. This phenomenon is still subject to extensive investigations but the trend is clear, fracture toughness

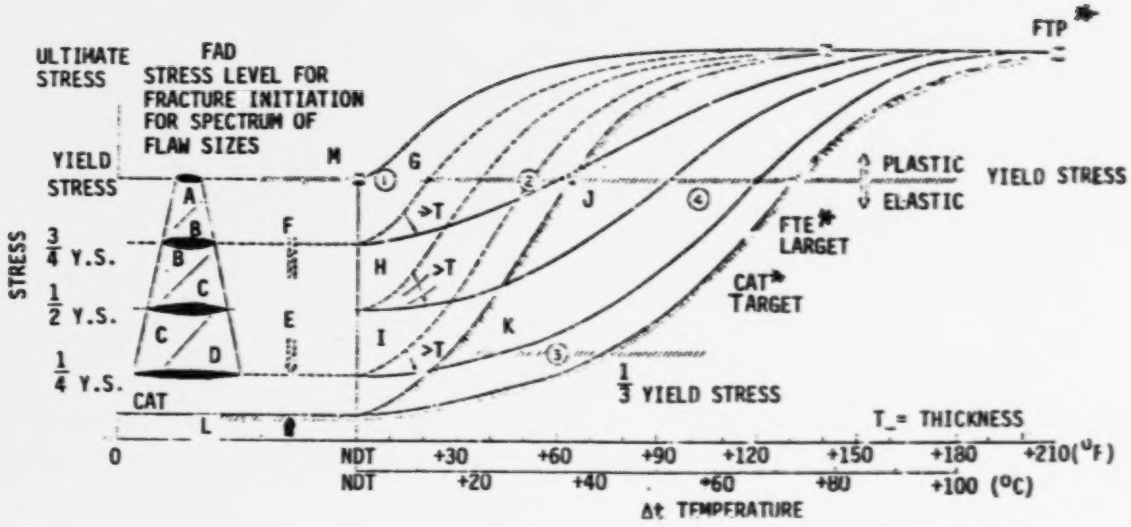

Figure 32 FAD expansion to include the effects of very large section size. Fracture-safe performance with respect to unstable fracture is indicated for temperatures in excess of the limits defined for the engineering design points noted as , 1) small flaws subjected to yield stress levels, 2) intermediate size flaws subjected to yield stress levels, 3) very large flaws subjected to nominal design stresses, and 4) very large flaws subjected to yield stress levels.(94) To obtain equivalent units in cm multiply the inches by 2.54, and the feet by 30.5. To obtain equivalent units in MPa multiply above values by 6.9. A) 4 inches, B) 6 inches, C) 12 inches, D) 2 feet, E) service failure experience small and large thicknesses, F) increasing flaw sizes (length), G) small flaw -common to all thicknesses, H) small thickness, I) large thickness, J) FTE-small thickness, K) CAT-small thickness, L) 5-8 ksi stress limitation, and M) not fixed all thicknesses.

* FTE - Fracture Transition - Elastic
CAT - Crack Arrest Temperature Curve
FTP - Fracture Transition - Plastic

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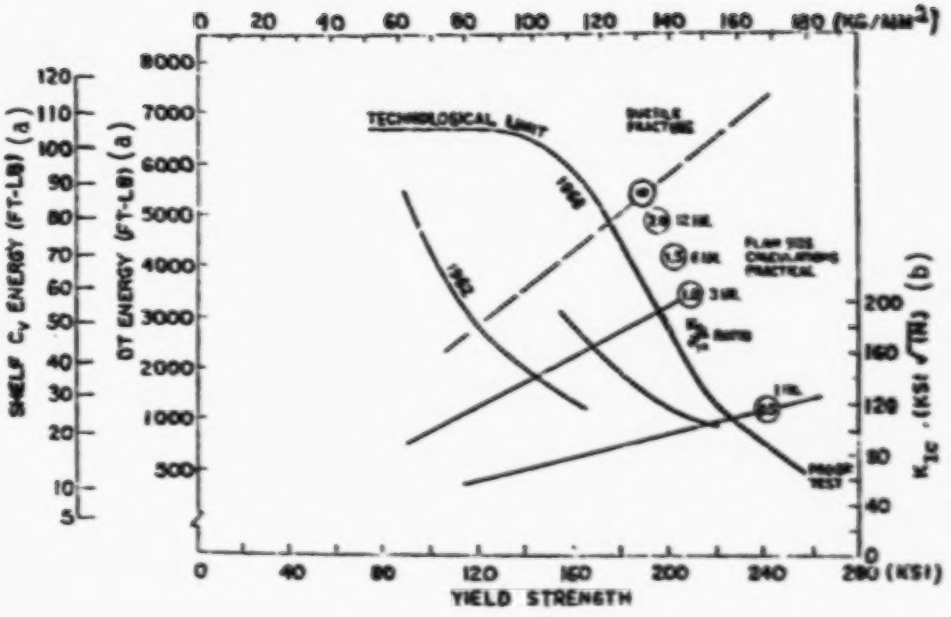

Figure 33 Ratio Analysis Diagram (RAD). The Grid of K Ratio Lines Indexes The Region Of The Diagram Which Relates To Plane Strain Fracture.(94)

(a) To obtain equivalent units in Joules multiply above values by 1.36.

(b) To obtain equivalent units in MPa $\sqrt{cm}$ multiply above values by 10.97

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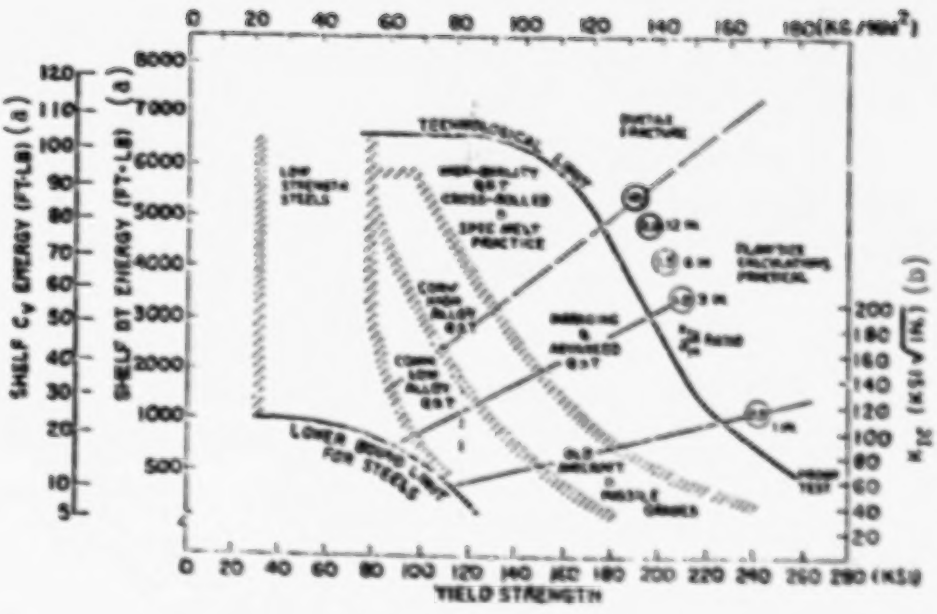
0 20 40 60 80 100 120 140 160 180(KS/IN²)
SHELF C_V ENERGY (FT-LB) (a)
SHELF DT ENERGY (FT-LB) (a)
TECHNOLOGICAL LIMIT
METAL FRACTURE
LOW STRENGTH STEELS
HIGH-QUALITY Q&T CROSS-ROLLED & ESR MELT PRACTICE
CONV HIGH ALLOY Q&T
CONV LOW ALLOY Q&T
HARDENING & REVENGED Q&T
OLD AMERICAN & BRITISH GRADES
LOWER BOUND LIMIT FOR STEELS
PLASTIC CALCULATIONS PRACTICAL
DROP TEST
K_{IC} (KSI $\sqrt{IN}$) (b)
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 (KSI)
YIELD STRENGTH

Figure 34 Metallurgical zoning of the RAD which defines the general effects of melting and processing factors on the strength transition. The three corridors of strength transition relate to metallurgical quality (void site density), which controls microfracture processes and, thereby, the macroscopic fracture toughness of the metal. The location of generic alloy steel types are indicated by the notations. (94)

(a) To obtain equivalent units in Joules multiply above values by 1.36.

(b) To obtain equivalent units in MPa $\sqrt{m}$ multiply above values by 10.97.

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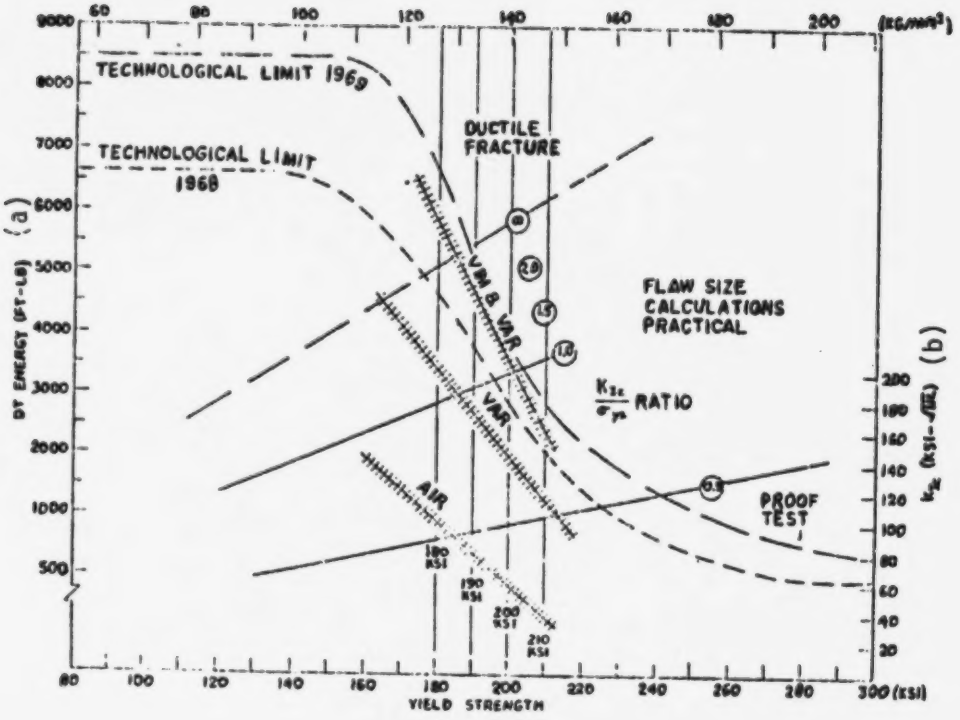

Figure 35 Metallurgical zoning of the RAD which defines the effects of specific melting practices. These include air melting under slags (Air), vacuum induction melting (VIM) and vacuum are remelting (VAR); a combination of these is indicated by the VIM-VAR notation. The extension of the Technological Limit curve to the 1969 position results from the use of advanced VIM-VAR melting practices at large production scale.(94)

(a) To obtain equivalent units in Joules multiply above values by 1.36.

(b) To obtain equivalent units in MPa $\sqrt{cm}$ multiply above values by 10.97.

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decreases with increasing section size for a given temperature within the transition region. In effect, increased thickness elevates the temperature regime of applicability of linear elastic fracture mechanics.

In a report on fracture toughness of a variety of shipsteels similar to A 36 type steel Hawthorne and Loss[91] estimate constraint elevation of transition type temperatures of 25 to 40°F (-3.9 to 4.4°C) with the thickness going from 1 in. (2.5 cm) to 2 in. (5.cm). The effect is incorporated into the familiar FAD diagrams shown in Figure 32. Nevertheless, such considerations do not address the spectrum of toughness values measured in commercial quality type plate. Differences in transition temperature behavior from plate to plate within each grade are detected to an extent that frequently obscures the large difference in composition from grade to grade, and even in heat treatment and deoxidation practice.

5.5.2 Fracture Control

The need for fracture control is illustrated by the King's Bridge Failure in Melbourne, Australia. This bridge contained 4 girders with one girder cracked to within a few inches of the top flange a year before the final bridge failure. Thus, structural redundancy allowed the bridge to re-distribute its loads and function well beyond the time a primary structural member had failed. By contrast, the failure of a primary member in the Silver Bridge at Pt. Pleasant, W.Va. resulted in complete collapse of the entire structure within a few seconds.[98]

To prevent catastrophic failures, three basic methods are usually considered:

Use of low design stresses

Use of tough steels

Prevention of large flaws in the structure.

A fracture control plan for a particular structure addresses these points and, conservatively, includes a structurally redundant component to postpone onset of catastrophic failure of the bridge.

Fracture control plans of specific types have now resulted in general specifications in some areas of application. The more advanced ones are based on fracture mechanics considerations.[101] Generally Nil Ductility Temperatures (NDT) are used in the design of the more basic structures. The American Society of Mechanical Engineers (ASME) pressure vessel codes generally require NDT's to be below the anticipated minimum operating temperature[102] while the Federal Highway Administration (FHWA) and American Association of State Highway and Transportation Officials (AASHTO) recently approved toughness requirements at temperatures corresponding to 70°F (21°C) above the minimum expected operating temperature for bridges.[103] The latter approach depends on the following:

a) Loading rate does not exceed 10^{-3} sec.$^{-1}$ Rate of loading has a pronounced affect on the ductile to brittle transition temperature as shown in Figure 36. The strain rate in a standard impact test is approximately 10 sec.$^{-1}$,[104] while the standard tensile test corresponds to less than 5×10^{-5} sec.$^{-1}$. [105]

b) Stress levels do not exceed the AASHTO maximum allowables which generally are less than 50% of minimum specific yield strength, for low alloy steels. Actually measured stresses in bridges have been reported to be less than 6 ksi (41 MPa) or about 10% of minimum yield strength.[103]

Unfortunately, relatively little information is published or otherwise available regarding the nil ductility temperature of structural steels.

A572 Grade 50 Steel (σ_y=50 ksi) (345 MPa)

● Slow Bend Load ($\dot{e}\simeq 10^{-5}$ sec^{-1})

■ Intermediate Strain Rate Load ($\dot{e}\simeq 10^{-3}$ sec^{-1})

▲ Dynamic Load ($\dot{e}\simeq 10$ sec^{-1})

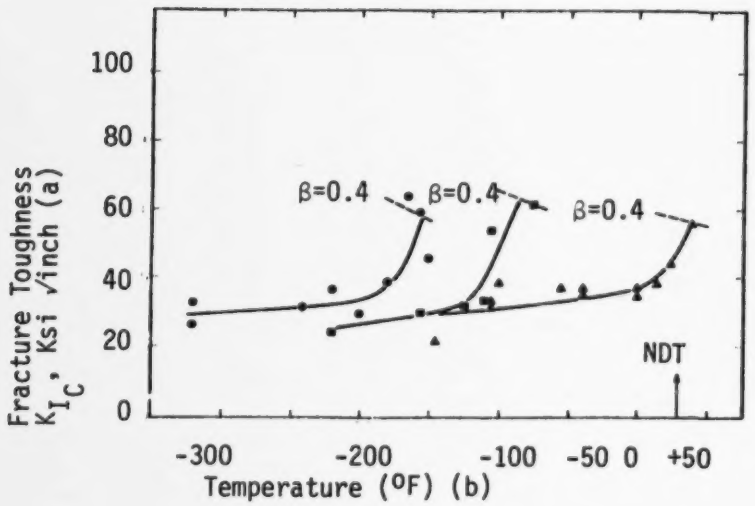

Figure 36 Effect of Temperature and Strain Rate On Fracture Toughness of A572 Grade 50 Steel.(101)

a) To obtain equivalent units in MPa √cm multiply above values by 10.97.

b) To obtain equivalent units in °C subtract 32.0 and multiply above values by .56.

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Attempts are now underway to develop correlations between NDT and the more readily available Charpy-V test data.[101,106]

NDT Specifications

Suess and Proft[101] have found a spread of NDT values for the same commercial steel grade (ASTM A572 - Grade 50) of 110°F, (43.3°C), indicating the strong effects of even subtle variations of microstructure on notch toughness making generalizations about "typical" nil ductility temperatures all but useless for design purposes. With these variations to be expected from plate to plate, these authors attempted to find a correlation between the more "economical" and "accepted" Charpy tests and the more useful Nil Ductility Temp of ASTM A572 Grade 50 steel for quality control purposes. They found that while it is impossible to accurately predict NDT from CVN results, a CVN energy value can be established at a particular temperature to indicate if the NDT is above or below that test temperature.

5.5.3 Design Criteria

The application of linear elastic fracture mechanics has primarily involved the ultra high strength steels. The fracture problems in structures of the lower strength steels frequently permit an LEFM analysis to be conducted, but an LEFM analysis applies strictly only when fracture can occur in essentially a brittle manner. Complete fracture toughness characterization, however, involves detailed knowledge of how the toughness increases in the transition region, and the service factors which include the minimum temperature, stress level, and constraint condition.[107]

Criteria used for fracture toughness in specifications include the temperature found at the 20 ft-lb. (27J) level, or the capability of the metal to bring 20 ft-lb. at the operating temperature. These are criteria which do not necessarily have to relate to a specific level of structural performance. Since a variety of products is usually covered in the specifications and they may be used in a wide range of structures, it is not practical for a toughness criterion to be quantitative in a general specification.

In view of the fact that welded structures below the transition region of the material are hazardous because of the small flaw size of critical pop-in flaws, the Navy has adopted the policy that a good minimum of toughness in highly stressed but not fracture critical structural members is the dynamic plane strain limit. This means the NDT of the steel should be below the minimum service temperature when sections are between 5/8 in (16mm) and 1.5 in (38mm) thick. For thicker sections, with potentially larger pop-in cracks, the plane-strain limit is some increment above the NDT temperature. For fracture critical members, the performance should be higher than the plane-strain limit, for example, $r + a\ \beta = 0.9$ level. This criterion provides better protection from undetectable flaws, and is a readily understandable structural performance

criterion.[107] This criterion implies that under tensile load with nominal load at 1/2 σ_y a flaw can grow until it penetrates the section before it becomes critical.

This large a flaw should be detectable before it becomes critical and initiates an unstable fracture, and any small pop-in flaw will be arrested. The Navy suggests that for complex structural details, such as welded intersections, the thickest section should be considered as the controlling thickness.

The use of steels in applications that require field welding for fabrication - such as in bridge construction - a reliable consistancy in degree of toughness must accompany the major mechanical property design criteria. Toughness relates to the ability of the material to absorb unexpected or undefinable stresses before fracture and depends fundamentally on these major factors:

Flaw Size

Stress Level

Ambient Temperature

Material Yield Strength

Nil-Ductility Temperature

Depending on the extent of which these factors are known, a number of methods are available to predict the onset of brittle fracture. To prevent brittle fractures, the combination of these factors must be held within limits.

5.5.4 Fatigue

The significance of avoiding flaws cannot be overstated in view of the possibility that a bridge is subject to load cycling which may cause an initially small and noncritical flaw to grow at various rates to critical size at which unstable crack growth might occur. Since elimination of all

weld defects is desirable but costly, the pre-existence of flaws must be assumed, and crack growth rate control becomes part of the fracture control plan. Conservative estimates, of fatigue crack growth per cycle of loading, da/dn, have been made for various structural steels.

$$da/dn = 0.66 \cdot 10^{-8} (\Delta K_I)^{2.25}$$

for martensitic steels

$$da/dn = 3.6 \cdot 10^{-10} (\Delta K_I)^{3}$$

for ferrite pearlite steels

where ΔK_I = stress intensity factor range

Rolfe [98] suggests integration of the crack growth expression between a_o (inital crack length) and a_{cr} (at K_{IC}) to obtain (the life time) prior to the crack reaching anticipated critical length (and expected failure).

$$N_P = \int_{a_o}^{a_{cr}} \frac{1}{C (\Delta K_I)^m} \, da$$

where N_P = number of cycles to reach the crack size a_{cr}, C and m are material constants with values as indicated in the previous 2 equations.

The problem resides in determining the initial crack length a_o. Several theories are proposed [109,110] to view the total fatigue life, N_T, of a component as the sum of its life to crack initiation, N_I, and the life propagation to critical crack length:

$$N_T = N_I + N_P$$

Hence N_P is not the whole fatigue life, and approaches it only if the stress range is far above the endurance limit. Near the limit N_T approaches N_P. N_I, that part of the fatigue life which relates to the initiation of the crack is related to

$$N_I = f\{K_f (\sigma S \sigma e E)^{\frac{1}{2}}\}$$

where K_f = the fatigue stress concentration factor of the notches in the specimen

σS= nominal stress range

σe= nominal strain range

E= the elastic modulus

Chew and Lawrence [109] have shown that the life-calculation method he proposes produces N_T values numerically equal to the values obtained through the rate calculation method advanced by Socie et al. [110]

The life calculation method enjoys two advantages: 1) it is easier and simpler to use; 2) it provides a nonarbitrary definition of a_o as well as predicting the fatigue life (N_T).

According to Barsom [111] fatigue life of structural components is determined by the initiation of subcritical cracks and their propagation to critical dimensions. At the critical size unstable fracture and failure of the component must be expected.

Since Fine [88] has traced the origin of cracks to the existence of dislocation interactions, a material must be generally considered flawed since there are no dislocation-free microstructures and particularly not in the commerical varieties of A36, A514 and A588 steels.

Consequently the growth rate of cracks is a basis for assessing component life with the weld to define an initial crack length.

Barsom showed that crack growth rates of A514 - B steel in bridges under actual load conditions can be simulated reasonably well in the laboratory with constant amplitude load fluctuations, where the amplitude is equal in magnitude to the root-mean square stress-intensity-factor range of that

Raleigh distribution function, which represents the actual variable load spectrum in the bridge. Figure 37 displays their results in terms of ΔK and Figure 38 in terms of ΔK_{rms}. This leads to an equation of the type

$$da/dn = A (\Delta K_{rms})^{n} \text{ where}$$

ΔK_{rms} is the root - mean square stress-intensity factor fluctuation (ksi$\sqrt{\text{inch}}$) or (MPA cm), A and n are constants obtainable from Figure 38.

Results of NCHRP research projects [112,113] on fatigue crack propagation in bridge steels indicate that a formalized random load crack propagation analysis can be related to actual bridge loadings.

Through-thickness loading is virtually unavoidable in most welding joints at right angles. Adams et.al[89] found that property changes - as far as fatigue is concerned - are such that surface properties are markedly superior to those at locations at ¼ or ½ in. (6 or 13 mm) thickness. The differences between ¼ and ½ in.(6 and 13 mm) thickness are very small.

SEM as well as optical investigations showed clear indications of fatigue cracks initiated from flat inclusions at the surface. (Figure 39). The authors found that the through-thickness fatigue strength was considerably lower than the longitudinal fatigue strength and did not lend itself to correlation with tensile strength as reliably as did the longitudinal fatigue strength. At any rate, weld metal without cracks and porosity is generally better in fatigue than the short transverse direction of the steel plate. Consequently, there appears to be no reason to specify welding electrodes stronger than the plate in T-joints.

5.5.5 Test Correlations

In an attempt to correlate the behavior of small laboratory specimens and complex welded structures of bridge members in A36 type steel Schilling et.al [114] reported that beams with cover-plate ends were able to reach

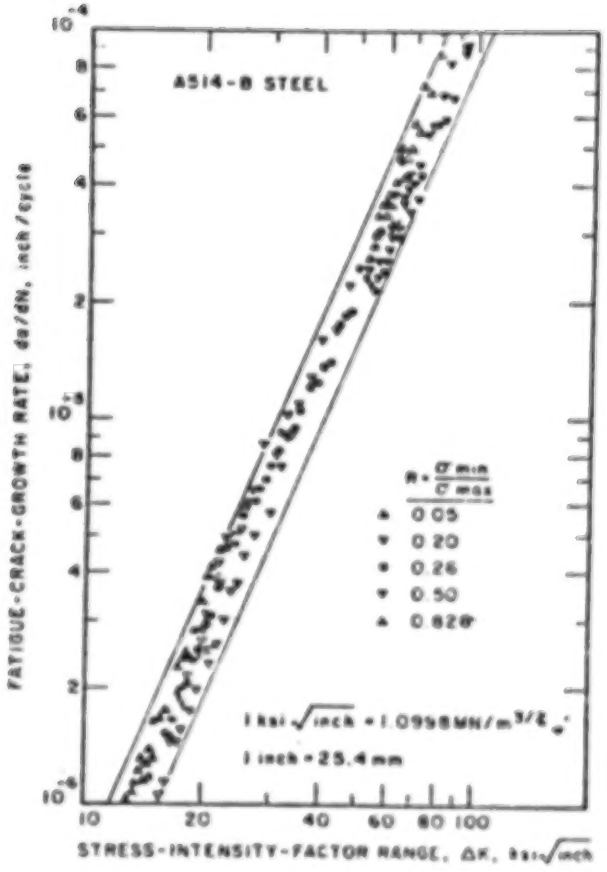

Figure 37 Crack-growth rate as a function of stress intensity range for ASTM A514 B steel. (111)

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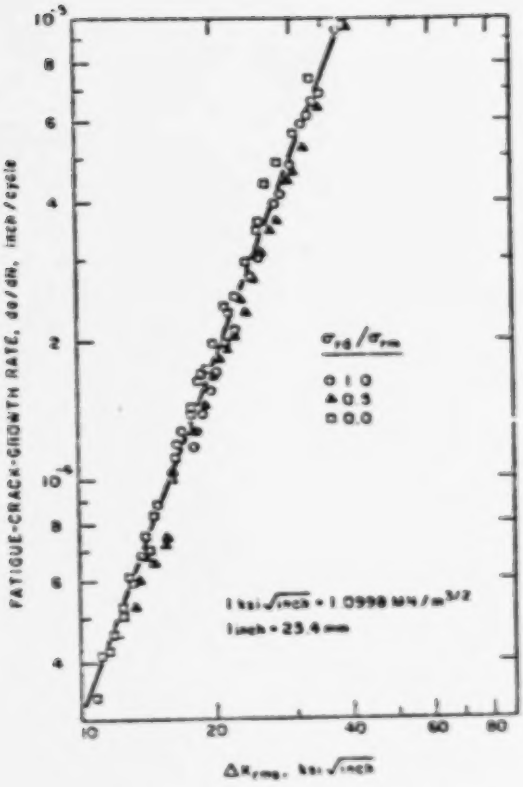

Figure 38 Crack-growth rate as a function of the root-mean-square stress intensity factor. (111)

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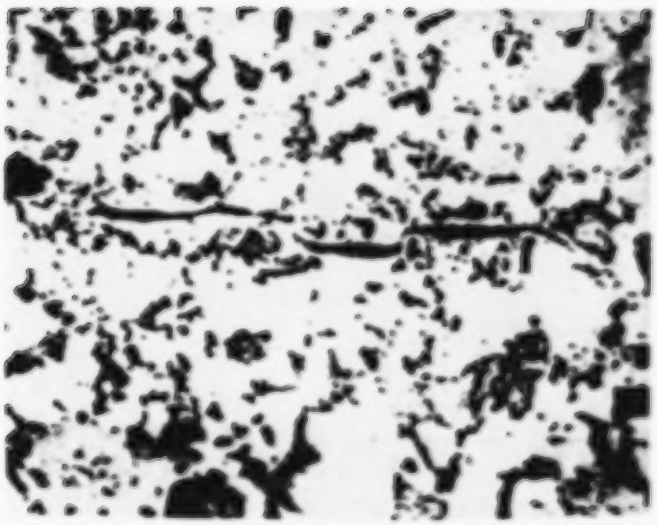

Figure 39 CROSS SECTION SHOWING CRACKING AT INCLUSIONS. MAGNIFICATION OF THE PHOTOMICROGRAPH =500x.

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nominal bending stresses close to the specified minimum yield points of the respective steels after they had sustained 100000 cycles with a stress range of 21 ksi (145 MN/M ^{2}), the cyclic loading conditions specified by AASHTO for the design of cover plate ends. Cracks of about 0.35 in.(9 mm) were observed. In service, they expected these cracks to grow rapidly causing a fatigue failure in a relatively short time regardless of the level of fracture toughness provided in the steel, e. g., the materials had effectively exhausted their fatigue life. Yet fracture toughness tests at -30°F (-34.4°C) still showed sufficient toughness to prevent premature failure. These inconsistencies illustrate the difficulty encountered by the designer in attempting to predict fatigue and fracture behavior of weldments from laboratory data, in the absence of adequate knowledge of the magnitude and distribution of residual stresses and stress concentrations in complex welded structural bridge features.

The interpretation of such unexpected behavior must be viewed in light of such additional data from Bognchi et.al. [115], which suggests that timely interruptions of cyclic loading rates tend to increase the fatigue limit up to 13%.

Restraining in excess of σ_y has a damaging effect on fatigue life of structural steels but to a varying degree. A36 weldments, for example, are not as sensitive to some tensile overstrains as A514 steel.[116]

Munse [117] found excellent correlation between the fatigue behavior of experimental tension-biased random loaded structural steel weldments and random loaded fatigue design criteria recently proposed.

According to several investigators, [112,113,149] design details and stress range are the overriding factors affecting fatigue behavior. Accordingly, a study revealed that a classification of design details is practical with respect to reasonable expectation of some fatigue life. A summary is shown

in Tables 21 and 22. Of particular significance is that basic material properties as they differ between A36 and A514 did not produce the expected difference in fatigue behavior. Also, fatigue studies on the various design details did not show an endurance limit for some as expected from standardized tests on standard specimens. This result should have drastic consequences on specifications for fatigue performance of bridges, and the incorporation of certain design details such as type E into future bridge design.

5.6 Testing

All aspects and premises on which a structural design is based should preferably be checked for applicability and accuracy. The corresponding tests must reflect the expected service performance, and their selection must therefore depend strongly on the designer's ability to identify completely structural performance requirements and the welder's anticipated ability to select and maintain suitable welding processes.

Judicious use of available specifications is found to greatly help in communicating quality requirements to the welder and to keep the efforts and costs of testing within reasonable limits.

Mechanical testing as per ASTM 370-76 applies also to weldments. Additional tests peculiar to assessing weld quality are e.g.:

a) Bending tests

to determine how much bending can be tolerated before a crack is initiated. Evaluation is rather subjective.

b) Test welds with a variety of shapes

o determine slag inclusions, porosity, and cracks.

Hardness tests appear to give good correlations with tensile strengths of both the short transverse and longitudinal directions of the plate. Combined with the relationship of the fatigue strength to the tensile strength,

Table 21

Fatigue Test Results

Set No.	S_{min} (ksi)[h]	S_{rm} (ksi)[h]	S_{rd}/S_{rm}	No. of Cycles[a] (Mil.)	Steel Type	Test Article[g]
1	0	10	0.0	15.3[b]	A514	Cover-plated beam detail A
2	0	10	0.5	—[c]	A514	Cover-plated beam detail A
3	0	10	1.0	2.11	A514	Cover-plated beam detail A
4	0	20	0.0	0.90	A514	Cover-plated beam detail A
5	0	20	0.5	0.49	A514	Cover-plated beam detail A
6	0	20	1.0	0.20	A514	Cover-plated beam detail A
7	0	30	0.0	0.12	A514	Cover-plated beam detail A
8	0	30	0.5	0.17	A514	Cover-plated beam detail A
10	10	10	0.0	2.55	A514	Cover-plated beam detail A
11	10	10	0.5	1.77	A514	Cover-plated beam detail A
12	10	10	1.0	0.84	A514	Cover-plated beam detail A
13	10	20	0.0	0.48	A514	Cover-plated beam detail A
14	10	20	0.5	0.27	A514	Cover-plated beam detail A
15	10	20	1.0	0.21	A514	Cover-plated beam detail A
16	10	30	0.0	0.10	A514	Cover-plated beam detail A
17	10	30	0.5	0.10	A514	Cover-plated beam detail A
19	10	60	0.0	0.01	A514	Cover-plated beam detail A
20	10	4	1.0	8.44[d]	A514	Cover-plated beam detail A
22	40	10	0.0	1.48	A514	Cover-plated beam detail A
23	40	10	0.5	1.04	A514	Cover-plated beam detail A
24	40	30	0.0	0.06	A514	Cover-plated beam detail A
25	40	30	0.5	0.05	A514	Cover-plated beam detail A
26	0	10	0.0	1.86	A514	Cover-plated beam detail B
27	0	10	1.0	0.93	A514	Cover-plated beam detail B
28	10	10	0.0	1.27	A514	Cover-plated beam detail C
29	10	10	0.5	1.17	A514	Cover-plated beam detail C
30	10	10	1.0	0.47	A514	Cover-plated beam detail C
31	10	20	0.0	0.23	A514	Cover-plated beam detail C
32	10	20	0.5	0.17	A514	Cover-plated beam detail C
33	10	20	1.0	0.10	A514	Cover-plated beam detail C
34	10	30	0.0	0.08	A514	Cover-plated beam detail C
35	10	30	0.5	0.07	A514	Cover-plated beam detail C
36	0	10	0.0	1.40	A514	Cover-plated beam detail C
37	0	10	1.0	0.73	A514	Cover-plated beam detail C
101	0	10	0.0	1.60	A36	Cover-plated beam detail C
102	0	10	0.25	0.83	A36	Cover-plated beam detail C
103	0	10	0.5	0.61	A36	Cover-plated beam detail C
104	0	20	0.0	0.15	A36	Cover-plated beam detail C
105	0	20	0.25	0.13	A36	Cover-plated beam detail C
106	0	20	0.5	0.10	A36	Cover-plated beam detail C
107	10	10	0.0	1.99	A36	Cover-plated beam detail A
108	10	10	0.25	1.78	A36	Cover-plated beam detail A
109	10	10	0.5	1.04	A36	Cover-plated beam detail A
110	10	20	0.5	0.21	A36	Cover-plated beam detail A
111	10	10	0.25	0.16	A36	Cover-plated beam detail A
113	0	40	0.0	0.02	A36	Cover-plated beam detail C
114	0	5	0.5	5.97	A36	Cover-plated beam detail C
115	0	10	0.0	1.61[e]	A36	Cover-plated beam detail A
116	10	10	0.0	0.97	A36	Cover-plated beam detail C
117	10	10	0.5	0.53	A36	Cover-plated beam detail C
501	0	10	0.0	15.20[f]	A514	Cover-plate specimen
502	0	10	0.5	3.95	A514	Cover-plate specimen
503	0	10	1.0	2.39	A514	cover-plate specimen
504	0	30	0.0	.24	A514	Cover-plate specimen
505	0	30	0.5	0.18	A514	Cover-plate specimen

(a) Log-average (anti-log of the mean of the logs) for three replicate tests unless otherwise noted. (b) Plus two beams unfailed at 10 million. (c) All three beams unfailed at 11 million. (d) Plus two beams unfailed at 20 million. (e) Plus one beam unfailed at 6.6 million. (f) Plus one specimen unfailed at 25 million. (g) Detail A= cover-plate ends not cross welded, web-flange welds made last; Detail B= cover-plate ends not cross welded, cover-plate welds made last; Detail C= cover-plate ends cross welded. (h) To obtain equivalent units in MPa multiply listed values by 6.9.(113)

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Table 22
Correlation Factor

Detail [a]	Steel	S_{min} ksi [e]	Correlation Factor, C [b] S_{rd}/S_{rm}=0.25	S_{rd}/S_{rm}=0.50	S_{rd}/S_{rm}=1.0	Combined [c]	Number of Tests [d]
Cover plate A	A514	0	--	0.070	0.453	0.378	18
Cover plate A	A514	10	--	0.271	0.356	0.360	21
Cover plate A	A514	0,10	--	0.228	0.414	0.355	27
Cover plate C	A514	10	--	0.140	0.419	0.364	24
Cover plate S	A514	0	--	0.217	0.250	0.275	15
Cover plate S	A514	10	--	0.180	0.296	0.285	21
Cover plate S	A514	0,10	--	0.177	0.270	0.270	36
Cover plate A	A36	10	0.211	0.434	---	0.395	15
Cover plate C	A36	0	0.508	0.470	---	0.452	24

(a) A = beams with cover-plate ends not cross welded and web-flange welds placed last;
C = beams with cover-plate ends cross welded last;
S = cover-plate specimens with cover plate ends not cross welded.

(b) Factor based on log N = A - Blog S_{re}, with $S_{re} = S_{rmt}\ CS_{rd}$, and combined test data for S_{rd}/S_{rm} = 0.0 (constant amplitude) and the value of S_{rd}/S_{rm} shown.

(c) Includes data for all values of S_{rd}/S_{rm}.

(d) Number of tests included in combined data.

(e) To obtain equivalent units in MPa multiply listed values by 6.9.

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the hardness tests offer a reasonable prediction of the fatigue strength of the parent metal. [89] While such a superficial test might be sufficient under highly defined stress conditions, more critical structural members require a detailed approach to assessing fracture toughness aspects of the material as well as the structure as a whole. Substantial amounts of data have been accumulated in the Damage Tolerance Design Handbook [118] including values of K_{ISCC}, the plain strain fracture toughness value as affected by corrosion. The well known fracture toughness value K_{IC} is strictly speaking - defined as the stress intensity at onset of unstable crack growth under plane strain conditions. With finite dimensions, as in this specimen, this condition is rarely approached indicating that large specimen dimensions are required which accounts for the rather complex and time consuming test procedures involved. In addition, standard K_{IC} determinations according to ASTM F399-20, F399-74, are made under close to static conditions indicating the limited usefulness of K_{IC} under more dynamic conditions. Efforts are underway to develop rapid, inexpensive substitute tests [119,120] or find correlations with other well known and easily obtainable data such as CVN data. [121] Stress intensity factors for different loading cases with the same mode (e.g., tension, bending, internal pressure, and thermal stress in mode I) [122,123,124] can be simply superimposed. However, cases that involve other loading Modes (I,II,III) have to be treated differently. [125]

CRACK ARREST

Under certain conditions, a fast growing crack can still be arrested. The damage will be immediately obvious and corrective actions can be taken to prevent a larger catastrophy. If dynamic effects can be disregarded, a reliable analysis can be made. Otherwise, analysis becomes complicated and appropriate testing should be designed. [126]

The approach to crack arresting takes on the following three forms, singly or in combination:

Increasing the crack resistance locally,

decreasing the crack driving force locally, and

diverting the crack direction where it can terminate in a less harmful way.

All these approaches are based on forcing a change in the stress intensity by guiding the unstable crack into different materials, geometries, or controlled residual stress conditions (e.g., strategically placed and oriented weldments).

5.7 Examples of Failures

The overriding importance of weld detail is shown in the following example:[127] A crack occurred at a butt weld T-junction of a longitudinal stiffener to the web section of an I-beam bridge girder as shown in Figures 40 , 41 , and 42 . Observations are summarized in Figure 41. Materials checked out well within specifications. The final conclusion was that substantial increases in fracture toughness would not have prevented this crack and girder behavior due to random truck loading that would have caused the web crack to spread in a stable manner until a critical length was reached. In general, the failure is thought to have occurred primarily because the stiffener was considered as an architectual attachment rather than a structural component. A full penetration weld would have eliminated the problem.

In short term tests, the differences in base metal properties for the common bridge steels tend to be secondary to such factors as stress range and weld detail. If, however, flagrant violations of the required chemistries occur such variations will show up in reasonably well fabricated structures. In the following example,[128] failure occurred in four interior beams of a

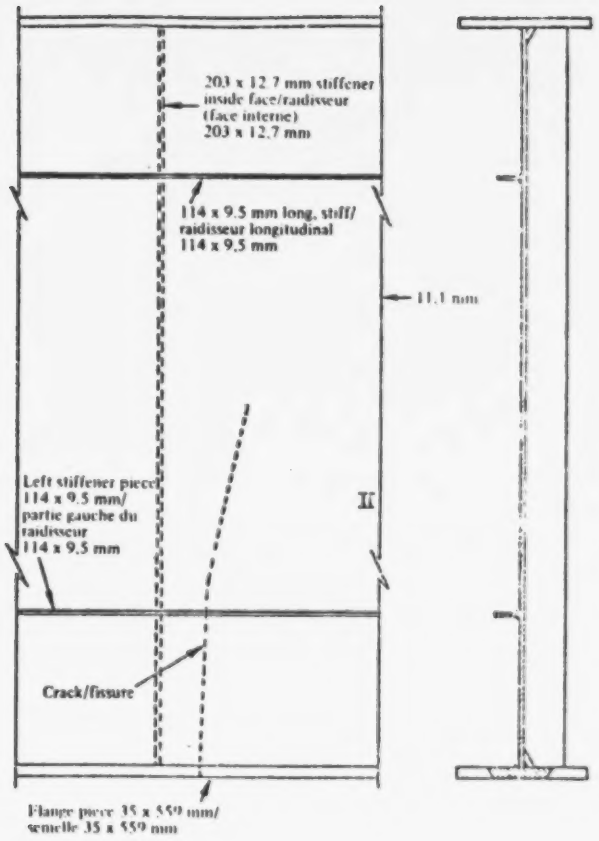

Figure 40 Schematic of Girder Showing Dimensions and Crack (127)
To Obtain Equivalent Units in Inches Multiply
above values by .04

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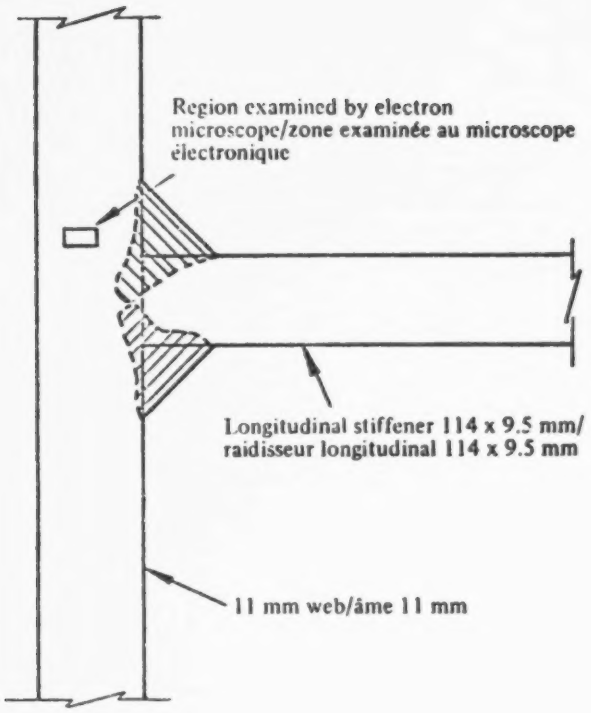
Region examined by electron
microscope/zone examinée au microscope
électronique
Longitudinal stiffener 114 x 9.5 mm/
raidisseur longitudinal 114 x 9.5 mm
11 mm web/âme 11 mm

Figure 41 Schematic Showing Location Examined By Electron Microscope(127)

To Obtain Equivalent Units In Inches Multiply Above Values By .04.

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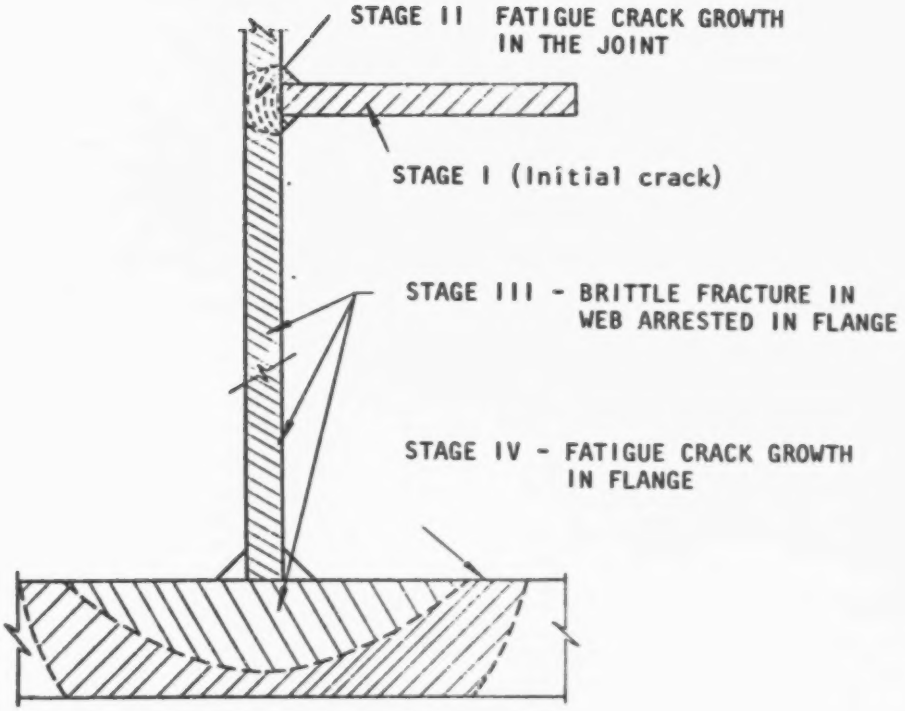
STAGE II FATIGUE CRACK GROWTH
IN THE JOINT
STAGE I (Initial crack)
STAGE III - BRITTLE FRACTURE IN
WEB ARRESTED IN FLANGE
STAGE IV - FATIGUE CRACK GROWTH
IN FLANGE

Figure 42 Schematic of Crack Growth States (127)

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7-6 box beam, a simply supported rolled beam and a concrete slab highway bridge. The fracture at the time of discovery extended 15.8 inches (400 mm) into the web of a beam, and 36 inches (914 mm) in depth as shown in Figure 43. The presumed causes of the accident were attributed to lower fatigue strength due to size effects, although various full size mechanical property tests did not reveal any problem with the base metal of the component. Chemical analysis, on the other hand, revealed a composition drastically different from the expected weldable grade of A242 (Reference Table 23 -for comparison the recommended ASM requirements are added). Carbon and manganese are both excessively high resulting in both, lower ductility and increased susceptibility to hydrogen cracking (Figure 44).

This case points out the importance of maintaining chemistry within specified limits, and the need for quality acceptance tests to eliminate welding the wrong material.

5.8 Recent Foreign Activities

A new concept in Russian Bridge building [129] is the so called"Bi-Steel-Method" which utilizes unalloyed bridge steels in highly stressed locations. Although stresses reach yield point proportions flow is prevented by high strength low alloy steel arresters. Material savings of up to 20% have been achieved in the case of a bridge across the Danube River in Germany. Similar attempts at bridge building for aesthetic as well as economic reasons are reported from Russia [130] and Germany.

Recent construction in Germany utilized the following steels [131, 132]:

1. St37, St37-2 for single track railroad bridges (Figure 45).

(Both steels are equivalent in composition to A282 type steels but the St37-2 requires fine grained practice in order to guarantee the minimum toughness level).

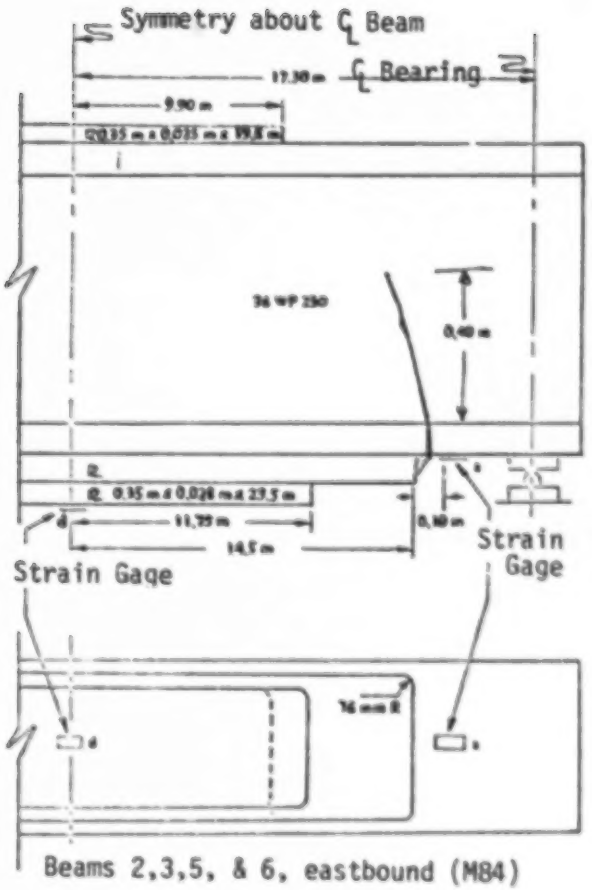

Figure 43 Location of Fracture. To Obtain Equivalent Units in Feet Multiply the Meter Values by 3.3, and to Obtain Equivalent Units in Inches Multiply the Millimeter Values by .04.(127)

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Table 23

CHEMICAL ANALYSIS OF STEEL (ASTM A242) [127]

	A242 Specifications/Specifications A242	
Carbon/Carbone	0.270%	0.26% max.
Manganese/Manganese	1.690%	1.30% max.
Phosphorus/Phosphore	0.027%	
Sulfur/Soufre	0.013%	0.063% max.
Silicon/Silicium	0.236T	
Copper/Cuivre	0.385%	
Chromium/Chrome	0.041%	
Nickel/Nickel	0.084%	
Molybdenum/Molybdene	0.019%	
Vanadium/Vanadium	0.002%	

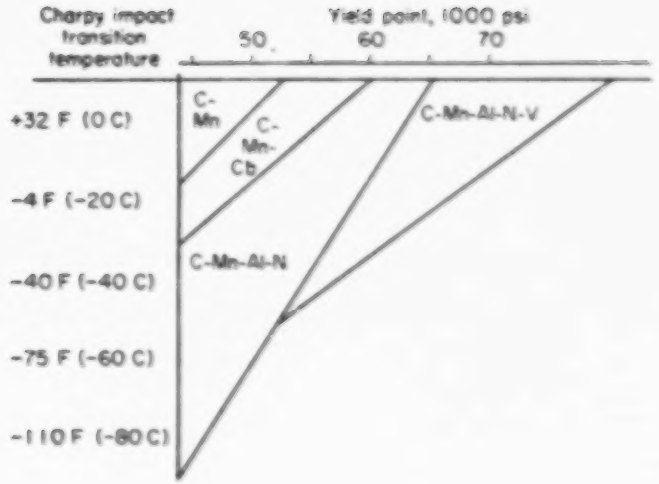

Figure 44 Possible combinations of yield strength and notch toughness available following normalizing fine-grained carbon-manganese steels with minor additions of suitable alloying elements. (Irvine and Pickering)

(Ref. Bain, C. E. and Paxton, W. H., "Alloying Elements in Steel", American Society for Metals, Metals Park, Ohio, 1966, p. 273)

To Obtain Equivalent Units in MPa Multiply X-axis Values by 6.9.

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View of the Cross-Section of the Super Structure

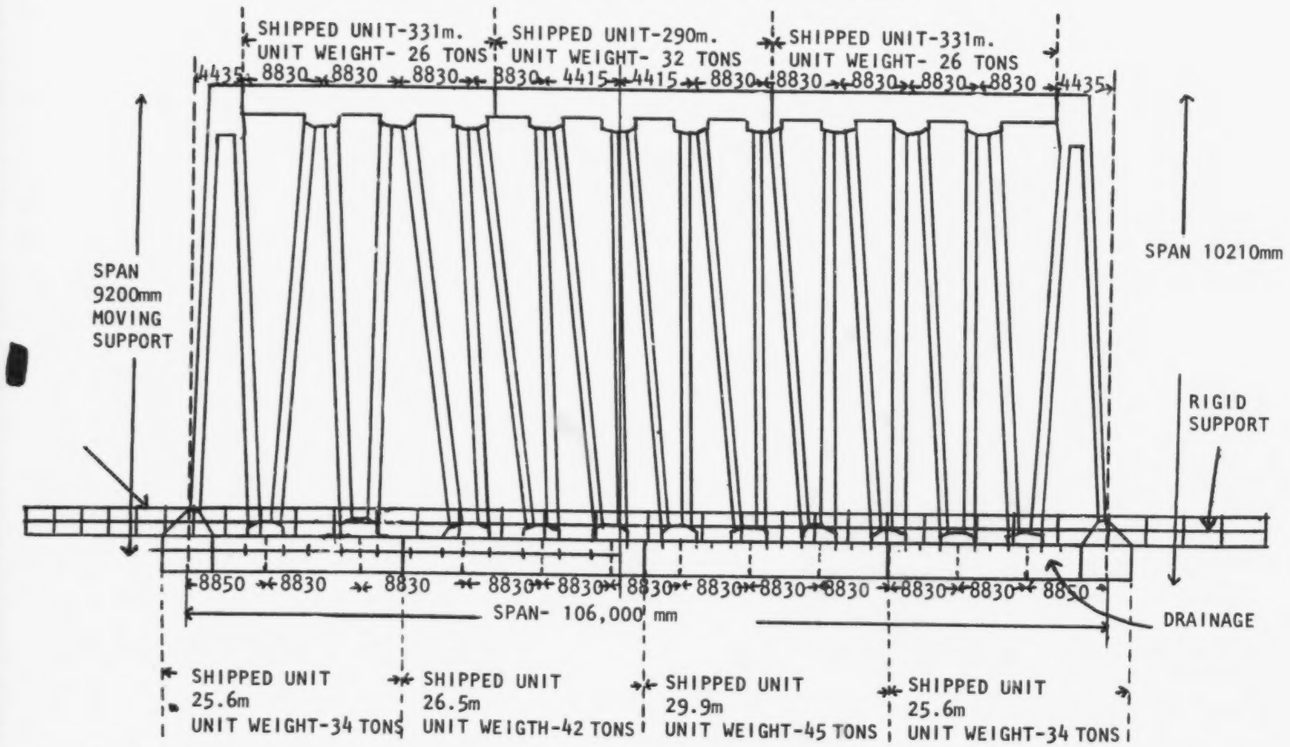
SHIPPED UNIT-331m.
UNIT WEIGHT- 26 TONS
SHIPPED UNIT-290m.
UNIT WEIGHT- 32 TONS
SHIPPED UNIT-331m.
UNIT WEIGHT- 26 TONS
4435 8830 8830 8830 8830 4415 4415 8830 8830 8830 8830 8830 4435
SPAN 9200mm MOVING SUPPORT
SPAN 10210mm
RIGID SUPPORT
8850 8830 8830 8830 8830 8830 8830 8830 8830 8830 8830 8850
SPAN- 106,000 mm
DRAINAGE
SHIPPED UNIT 25.6m UNIT WEIGHT-34 TONS
SHIPPED UNIT 26.5m UNIT WEIGTH-42 TONS
SHIPPED UNIT 29.9m UNIT WEIGHT-45 TONS
SHIPPED UNIT 25.6m UNIT WEIGHT-34 TONS

Figure 45 Overview and Truss Elements of a 106m (348 ft) Long Truss Bridge Across the RMD-Canal (Dimensions in mm). To Obtain Equivalens Units in Inches Multiply Above Values by 0.04/

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and, 2. St37 equivalent to A283 steels

St37-2 equivalent to A283 steels

St52 -3 equivalent to A 36 type steel.

These steels were selected for some highway bridges with the St52-3 in the most critical sections(Figures 46 and 47) requiring an elevated degree of corrosion resistance. [133] Particular attention was given to weld sequencing.

The effect of plate thickness on residual stresses in weldments has been described in a number of publications. [134,135] Methods are defined to predict the degree of deformation of the structure.[136,137]

It is generally recognized that K_{IC} values describe susceptibility to brittle fracture more accurately than any other test values. Simultaneously, it is realized that K_{IC} determinations are time consuming and costly which has lead to a considerable effort to find correlations with more readily available or obtainable properties such as C_v data. [138,139,140]

The concern of German welders with optimization of microstructures in the weldment and near-by base metal zones of high-strength steels has led to recommended procedures which include precise determination of crack propensity by calculation based on chemical analysis, welding rod composition match-up, welding techniques, and consideration for varying thickness. Pre-heat of heavy-gauge material to 212-392^{o}F (100-200^{o}C) is recommended as a rule to prevent cracking.[141] The detrimental effects of sulfur in base metal and weldment are known and research is underway to control amount, size, and shape of inclusions. [142]

Kawaguchi et. al. [143] have shown in some recent work that lowering the silicon level from the customary 0.3% to about 0.1% and/or adding boron in judicious amounts depending on the nitrogen level can lead to a distinct lowering of the transition temperature (Figures 48 and 49). Although

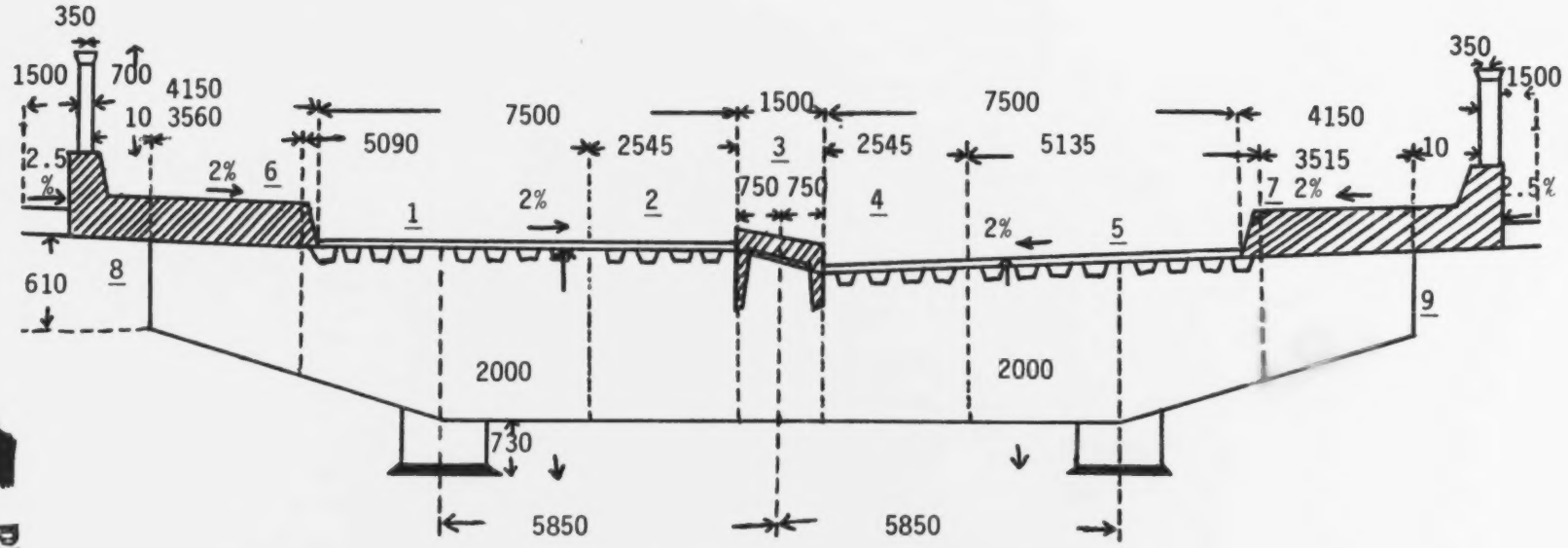

Figure 46 Cross-section and Construction Elements for Road Bridge "Gaterweg":(131
Bridge Elements 1 and 5: Main Girders, Bridge Elements 2 and 4: Interior Steel Deck Plate, Bridge Elements: Central Strip. (Dimensions in mm). To Obtain Equivalent Units in Inches Multiply Above Values by .04.

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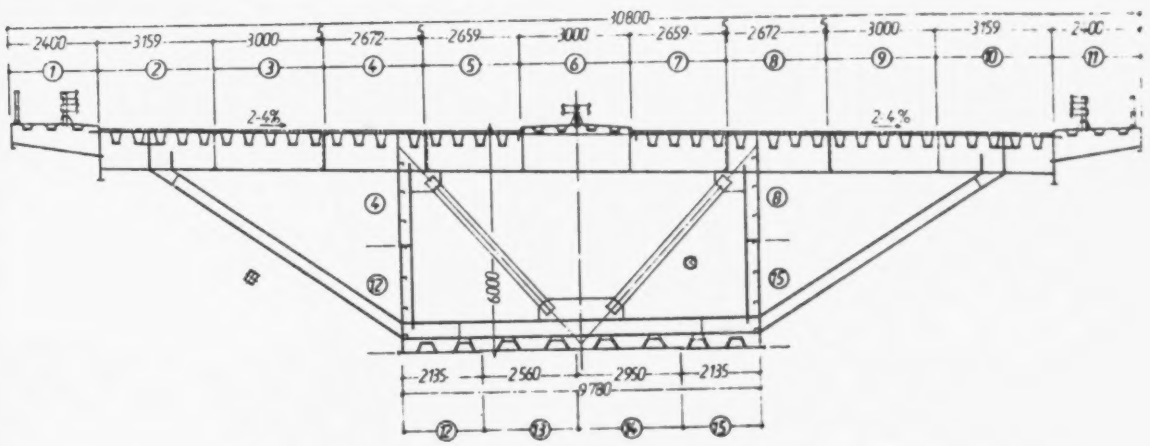

Figure 47 Cross-section and construction elements for Danube Bridge of Schalding: (131)

Bridge elements 1, 11:
side walks

Bridge elements 2, 3, 5, 7, 9, and 10:
road bed steel decks

Bridge elements 4, and 8:
main girders with road bed

Bridge element 6:
central strip

Bridge elements 12, and 15:
main girders with floor plate

Bridge elements 13 and 14:
floor plates

(Dimension in mm) , To Obtain Equivalent Units in Inches Multiply Above Values By .04.

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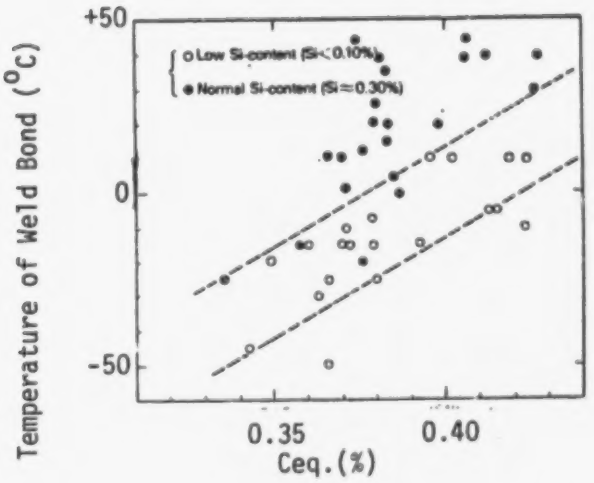

Figure 48 Effect of lowering Si-content to the toughness of the weld bond in large heat input welding (85.3 ksi (60 Kg/mm^2)) grade tensile strength steel plate quenched and tempered, 1.2 inches (30mm) thick, 2.4 x 10^5 Ft-lb/in. (130 KJ/cm)). To obtain equivalent units in ^{o}F multiply ^{o}C values by 1.8, and add 32.0.

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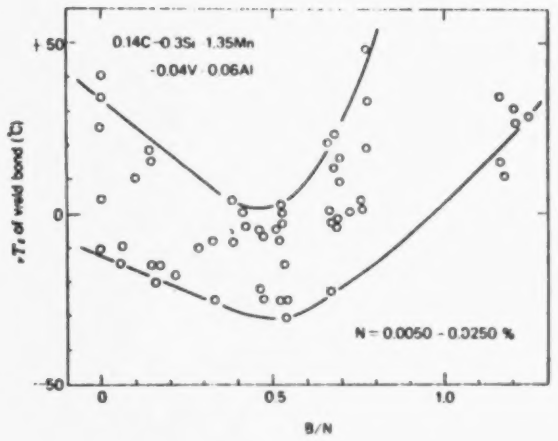

Figure 49 Effect of adding B to the toughness of the weld bond in large input welding (85.3 ksi ($60Kg/mm^2$)) grade tensile strength steel plate quenched and tempered. .5 in (13mm) thick, 2.4×10^5 Ft-lb/in. (130 KJ/cm)). To obtain equivalent units in ^oF, multiply ^oC values by 1.8 and add 32. (143)

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this data was generated using electro-gas welding, this still is representative of a high heat input process similar to submerged arc welding conditions.

A Russian investigation [144] coincides with the Japanese reports on improvement of resistance to brittle fracture in weldments by reducing the silicon level from the traditional 0.3% down to 0.1 - 0.2%. (Figures 50 & 51). In these figures, Steel 1 (15G2-AFDps) is the improved version of the other three steels. To maintain strength, the loss in silicon is compensated for by raising the copper level to 0.2-0.4%.

The kinetics of hydrogen redistribution in welded joints have been analyzed by Karhin et. al.,[144] indicating that while the highest concentrations will occur in the weld metal, some hydrogen will enter the heat affected zone, a region of high residual stress on cooling.

The HAZ is susceptible to cold cracking to a degree that depends on the microstructure which - in turn - depends on cooling rate and composition. Quenched and tempered materials are found to be tougher than controlled and normalized materials. [145]

German Specification writing bodies concern themselves presently with the adaptation of the concept of damage accumulation in assessing the life time of structures. In addition, some specifications contain new procedures for calculating the upper limit of static loading based on exposure to a fixed number of allowable load cycles (e.g., 60000).[146,147] Other work in preparation for specification concerns fatigue of weldments under simultaneous torsion and bending. [148]

Work by Ofsuka [150] has indicated that the toughness of a weld is determined by the fracture initiation behavior of each part of the weld, i.e., weld metal, HAZ, and base metal. The concerted behavior of these weld components has been the subject of development of a test for remanent ductility in welded joints. The resulting test is now being used as acceptance criterion in welded structures. [114]

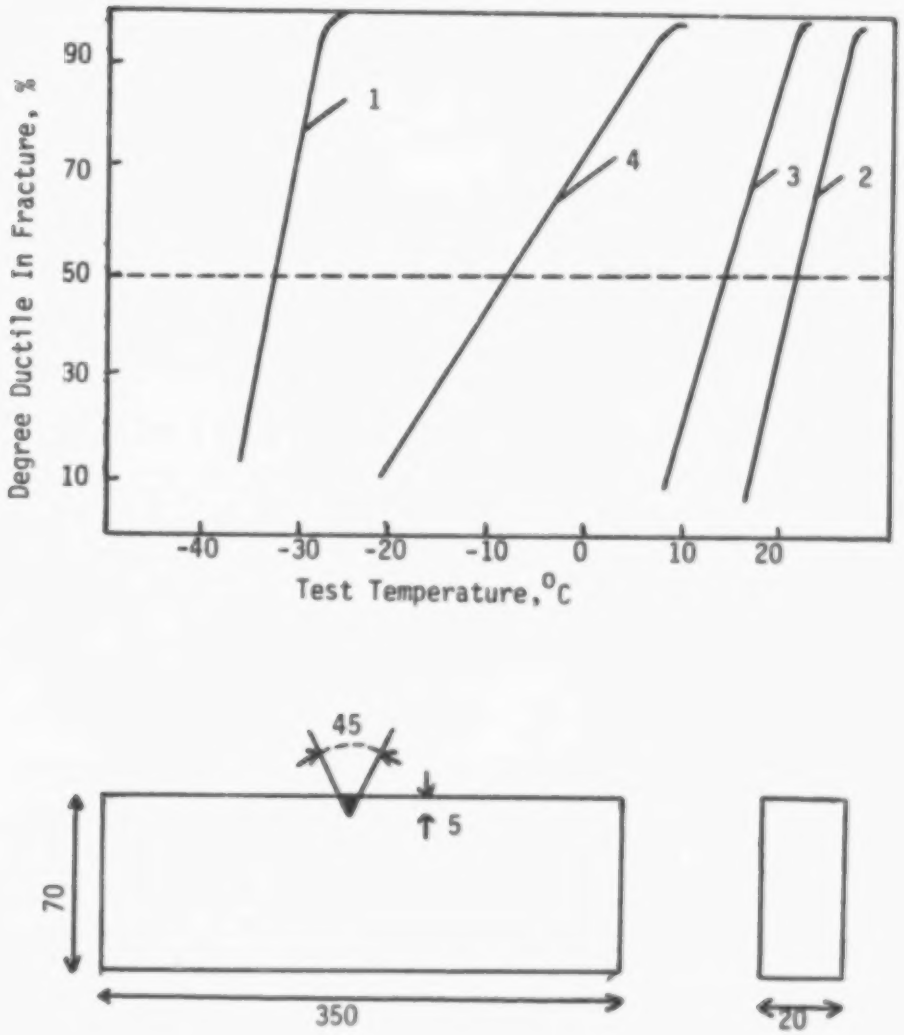

Figure 50 Degree Ductile Fracture of Specimens With Pressed-in Notch Versus Test Temperature (sheet 0.8 in. (20mm) thick): (144)

1) 15G2-AFDps;
2) 10KhSND;
3) 10G2S1;
4) 16KhSND

To obtain equivalent units in °F multiply the °C values by 1.8 and add 32. To obtain equivalent units in inches multiply mm values by .04.

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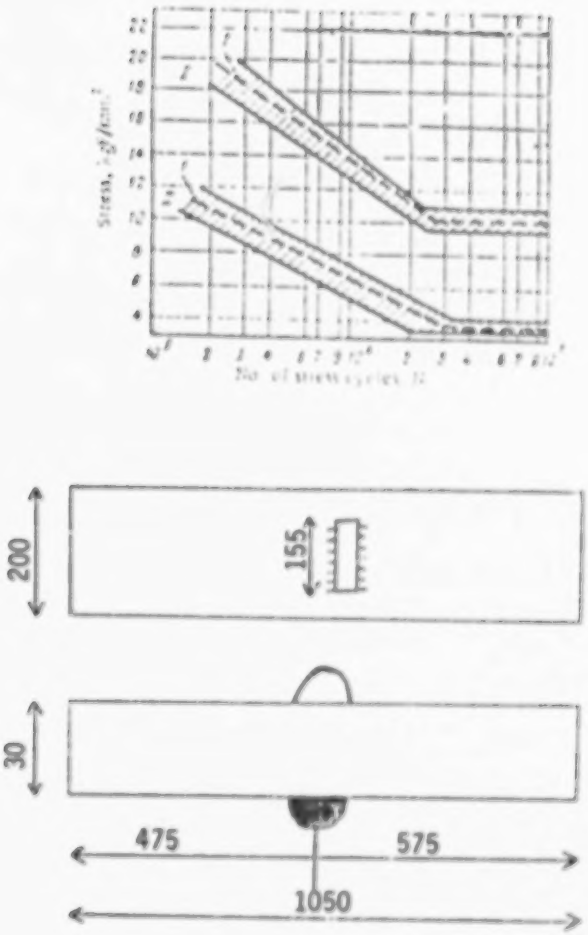

Figure 51 Endurance of Specimens Having A Transverse Angular Weld Of Semikilled Steel 15G2-AFDps (line) With Low-Alloy Killed Steel 10kh SND, 10G2S1 And 16 kh SND (hatched areas) In The Case Of Pulsed (I) And Symmetrical (II) Stress Cycles.

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Section 6

EXAMPLES OF INTERNATIONAL BRIDGE WELDING PRACTICE

6.1 USSR Bridge Fabrication Practice

The first USSR highway bridge made from low-alloy high-strength steel having an orthotropic deck was built in 1973[151] in Kamenetsk-Podolsk (Figure 52).

The main dimensions of the bridge are: the span -180m (600 ft.), the width -18.5m (61.6ft.), the weight -1150 tons. The 14Ch2GMR steel was used for the lower heavier loaded members, the rest of the members were made from 14G2 steel. Chemical composition of the steel is given in Table 24 along with the composition of the ASTM A514 steel which is included for comparison. As Table 24 shows, the quenched and tempered 14Ch2GMR steel is quite similar to the ASTM A514 steel. The filler metal featured extremely low carbon contents not exceeding .09% and was alloyed with Cr, Ni, and Mo. The chemical composition of the filler and weld metals is given in Table 25.

The plates were cut by oxyfuel cutting (not allowed by AWS Code) and the edges with a hardened layer were removed by machining. The surfaces to be deposited on plus 20mm (3/4 in.) material on both sides of the welds were brushed clean. The gap of the butt joints for the plate thickness of 12-14mm (.48-.56 in.) was 1-2mm (.040-080 in.).

The tack welds were performed by an electrode 4mm (5/32 in.) in diameter. The amperage was held between 160-180A. The butt welds were made by SAW on a flux backing with the parameters shown in Table 26.

Preheat and interpass temperatures were 60-80°C (140-180°F)* and 150-200°C

* *AWS Code for the ASTM A514 steel of this thickness range recommends minimum preheat temperatures of 10-93°C (See Table 3-5).*

Figure 52 Bridge in Kamenetsk-Podolsk (USSR)

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TABLE 24

CHEMICAL COMPOSITION OF BRIDGE STEEL FROM USSR (151) AND USA

Grade of Steel	C	Mn	Si	S	P	Cr	Mo	B	Ti
14G2	.12-.16	1.4-1.8							
14Ch2GMR*	.13-.16	1.1-1.2	.29-.32	.024-.033	.017-.030	1.4-1.6	.45-.54	.004-.0056	.026-.031
ASTM A 514 Grade L	.13-.20	.40-.70	.20-.35	.04	.035	1.15-1.65	.25-.40	.0015-.005	.04-.10

Note: * Used after quenching at 1706-1742°F (930-950°C) in water and tempering at 1256-1292°F (680-700°C). Yield strength = 60-82 Kg/mm^2 (85-116 ksi), Tensile strength = 70-92 Kg/mm^2 (99-130 ksi), elongation = 14-22%, Cv-notch toughness = 4.0-9.5 $kg_f \cdot m/cm^2$ (0.4-0.9 ft-lb) at -40°F (-40°C).

TABLE 25

CHEMICAL COMPOSITION OF FILLER METAL [151] .

METAL	COMPOSITION								
	C	Mn	Si	Cr	Ni	Mo	S	P	Al
WIRE Sv-08ChN2M	0.08	0.55	0.28	0.77	1.57	0.29	0.023	0.017	
WELD	.10-.12	.83-.89	.30-.35	.87-.91	.72-.76	.39-.42	MAX .02	MAX .021	
WIRE Sv-08ChN2GMY	0.09	1.20	0.47	0.76	2.0	0.65	0.021	0.018	
WELD	.07-.10	.85-.95	.30-.33	.82-.92	1.22-1.32	.47-.55	MAX .020	MAX .022	MAX .15
WIRE Sv-D8GA	0.07	0.94	TRACES	—	—	—	0.025	0.015	
WELD	.08-.10	.80-.85	MAX.20	MAX.6	TRACES	MAX.18	MAX .022	MAX .019	

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TABLE 26

WELDING CONDITIONS FOR BRIDGE FABRICATION (151)

Joint	Thickness		No. of Pass	Welding Conditions				
				Amperage	Voltage	Travel Speed		Polarity
	MM	IN		A	V	M/HR	IN/MIN	
Double Square Groove	12	0.5	1,2	540-560	35-37	27.5	18.0	DCRP
	14	0.6	1,2	600-620	36-38	32.0	21.0	DCRP
Double-V-Groove	18	0.7	1	440-460	35-36	18.0	11.8	DCSP
			2,3	500-520	35-37	21.5	14.1	DCRP
	28	1.1	1	460-500	36-37	18.0	11.8	DCSP
			2,7	550-580	36-38	21.5	14.1	DCRP

(300-390°F), the latter being kept for 30-40 min after the welding has been completed. The relatively low preheat temperature was selected instead of the recommended 302-392°F (150-200°C) because it was noticed that in the course of fabrication of the members more than 2m (6.6 ft.) long, the higher preheat temperature caused considerable transverse shrinkage and under unfavorable conditions, (small width-to-depth weld ratio, higher carbon content in the base metal and so on) the plates' movement may initiate formation of longitudinal cracks in the first layer. That is why in order to improve width-to-depth ratio, and to reduce base and weld metal admixture, the straight polarity (DCSP) was used for the first layer of the plate 18-25mm (0.7-1.0 in) thick. In conjunction with a low carbon filler metal (09GA alloyed only with 1% manganese (see Table 25))it provided crack free weld metal for the first layer.

The welding was performed continuously. When interruption occurred, the temperature of the weld metal was kept heated not lower than 80°C (180°F). The direction of each pass was opposite to the subsequent one to obtain uniform heat input along the weld. The welding was performed at an ambient temperature not lower than 5°F (-15°C).

The preliminary tests showed mechanical properties of the weld equal to the base metal (Table 27).

The plates of the lower part of the main girder were joined with the longitudinal stiffeners and vertical webs by both-side two-pass fillet welds using semi-automatic CO_2 metal arc welding with parameters: 280-300A, 28-30V, 12m/hr (39.4 ft/hr). Two passes were needed to eliminate the hard structure in the HAZ by reheating the weld metal of the first layer.

Visual inspection, ultrasonic inspection of all the bridge welds, and X-ray inspection of the weld intersections and the weld terminations were conducted on all butt welds. Unacceptable defects were all types of cracks,

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TABLE 27

MECHANICAL PROPERTIES OF THE WELDED JOINT (151)

THICKNESS mm (a)	YIELD POINT kgf/mm^2 (b)	TENSILE STRENGTH kgf/mm^2 (b)	ELONGATION %	CV-NOTCH TOUGHNESS kgf.m/cm^2 (c)			
				+20°C (d)	-20°C	-40°C	-60°C
12,14	63.6	78.8	19.2	10.2-12.0 / 11.0	8.6-11.0 / 9.5	8.2-9.3 / 8.5	5.3-7.8 / 7.2
18	71.3	79.9	20.2	12.3-13.6 / 12.7	11.0-11.8 / 11.3	8.0-9.4 / 8.5	7.4-8.4 / 7.9
28	69.4	84.1	19.8	130-15.0 / 14.3	—	9.8-10.8 / 10.5	—

a) To obtain equivalent units in inches multiply listed values by .04.

b) To obtain equivalent units in ksi multiply listed values by 1.4.

c) To obtain equivalent units in Ft-lb multiply listed values by .09.

d) To obtain equivalent units in °F multiply listed values by 1.8 and add 32.

lack of fusion, lack of penetration, undercuts deeper than 1mm (.040 in.) single porosity and inclusion more than 1mm (.040 in.) in diameter and if the number of the latter within the area of 400mm (16 in.) long exceeded 4, and the distance between them was less than 50mm (2 in.).

When the article was published the bridge was successfully in operation for more than 3 years.

6.2 Standardization of the Bridge Fabrication (Germany)

The standardization of bridge construction parts makes possible rationalization measures in fabrication. This pre-supposes understanding and collaboration between designer, fabricator, and customer. Investments are considerable and to secure economic success it requires full utilization of equipment capacity. The investment costs must be related to increases in output. For fillet welds 4mm(1.55 in.) in size, the following welding speeds can be obtained on the basis of 100% arcing time:

a) manual welding - 20-25cm/min (8-10ipm)

b) semi-automatic GMAW - 40-45cm/min (16-18ipm)

c) fully automatic SAW with two six welding heads - 100-300cm/min (40-120ipm)

Standardization, in addition to a considerable saving, enables the fabricator to achieve a higher degree of mechanization, which results in construction of constant quality.

6.2.1 Design

Below is an example from German bridge fabrication practice. Figures 53 and 54 show the basic construction of two orthotropic bridges over the Danube near Schalding, and over the North Sea-Baltic Canal to the Radar Island, respectively. The design of the orthotropic decks of the bridges is shown in the Figure 55a. To achieve a standardization of the cross section

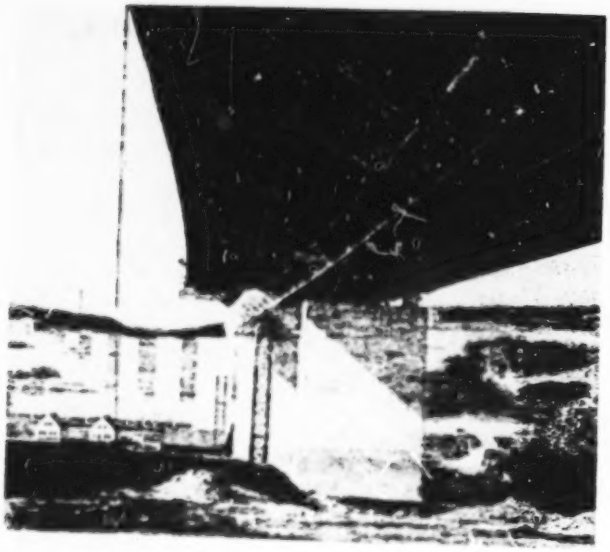

Figure 53 Bridge over the Danube near Schalding During the Construction.

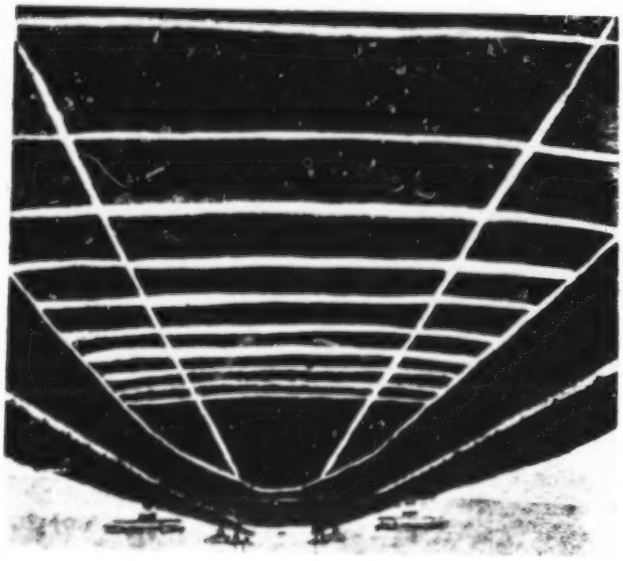

Figure 54 View of the Orthotropic Deck of the Bridge over the North Sea-Baltic Canal to the Radar Island.

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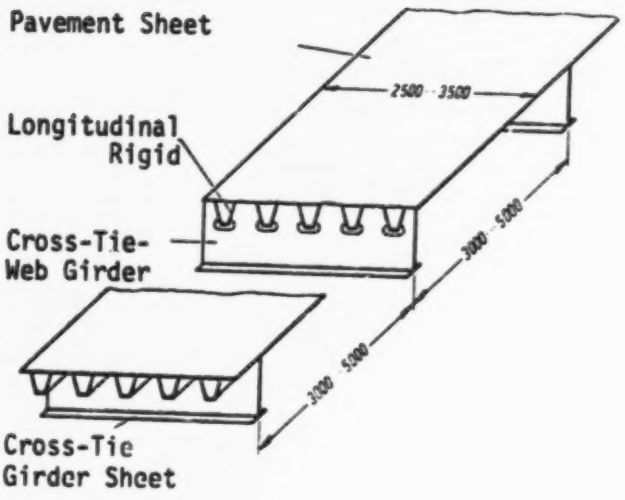
(a)



(b)

Figure 55 Design Scheme of Orthotropic Bridge Deck (a) and Longitudinal Stiffeners (b) of the Deck. Dimensions in mm. To obtain equivalent units in inches multiply above values by .04.

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of the decks, hollow trapezoidal stiffeners have been developed (Figure 55b) which satisfy all conditions of structural analysis and construction. The cold profiles are very accurate and deviations from the cross-sectional dimensions lie at 1 to 2%.

6.2.2 Assembly and Welding of the Deck Plate and Longitudinal Stiffeners

The constant pitch between longitudinal stiffeners and standardized stiffeners in profile, make it possible to use a jig in the assembly (Figure 56) which accurately adjusts the pitch of the stiffeners by its hydraulic swivel arms. The parts are joined by tack welding after pressing the hollow sections to the plate.

Subsequently, submerged arc welding using 6 welding heads operating simultaneously is carried out in a portal (Figure 57). Each welding head is guided by two touch-guide rollers which compensate for unevenness of the plates and curvature of the stiffeners. The deep penetration that submerged arc welding achieves, in addition to the required fillet weld allows approximately 2/3 of the hollow section wall thickness to be fused which secures against fatigue caused by wheel loads on the deck.

The welding parameters are: wire type S1, wire diameter 3mm (.125 in.), flux 10 by 498, 400-500A, 28-30V, speed 50-60cm/min (20-24ipm).

Welding produces distortion in the transverse direction as a result of angular shrinkage and curvature in the longitudinal direction. The maximum distortion permitted was 4mm (.155 in.) and the excessive distortion was reduced by flame straightening to the permitted values.

6.2.3 Assembly of the Cross Girders

The cross girder parts are needed in large numbers with only small dimensional differences between them. About 300 cross girder pieces are required for 1000 tons of bridge weight.

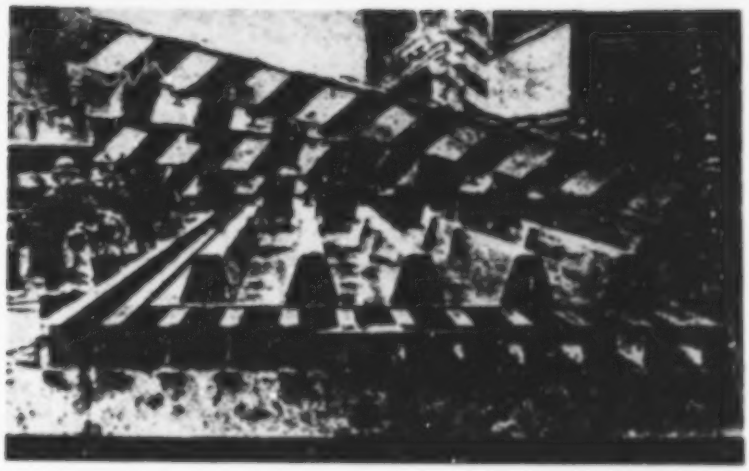

Figure 56 Assembly Jig for the Attachment of Trapezoidal Hollow Stiffeners to the Deck Plate.

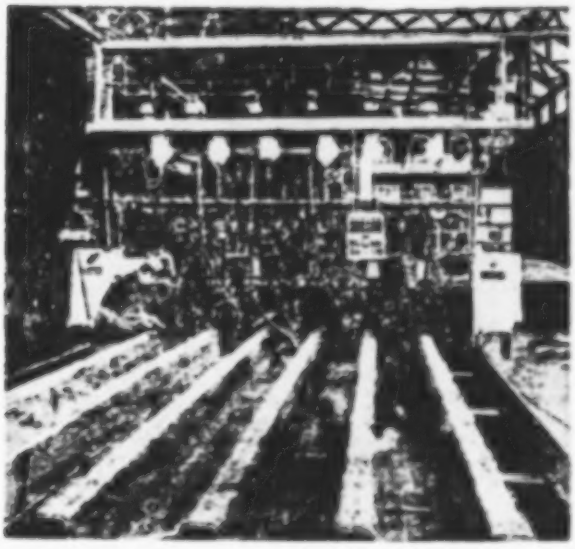

Figure 57 Portal with 6 Welding Heads for Submerged Arc Welding of Longitudinal Stiffeners to the Deck Plate

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The welds are relatively short (2.5-3.5m or 8 -12 ft.) for the application of fully automatic welding. For this reason, semi-automatic gas metal arc welding was used (Figure 58). The welding parameters are: wire type DIN8559, wire diameter 1.2mm (.045 in.), shielding gas CO_2 or a mixture with Argon, 250A, 30V, speed 25-30cm/min. (10-12imp).

6.2.4 Assembly and Welding of the Complete Deck

The prefabricated parts plate and longitudinal stiffeners as well as cross girders, are assembled in a working area specially set aside. The deck plate is clamped to a flat girder grid. This simplifies the placing of the cross girders. After tacking the cross girder webs, the plate is placed into a manipulator for the completion of welding. The mobility of the manipulator enables all welds to be done in the horizontal position (semi-automatic with shielding gas). Welding equipment provided with push-pull arrangements have proved successful, with these the welder can reach all weld locations without lengthy intervals. The welding parameters correspond exactly to those used in the fabrication of the cross girders (Section 6.2.3).

Figure 58 Semi-automatic Gas Metal Arc Welding of Cross-Girders

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GOVERNMENT PRINTING OFFICE: 1982 522-046/8521 1-3

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

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This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

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* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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